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SING SYSTEMS FOR MEASURING MECHANICAL PROPERTIES IN GROUND MASSES

Vol. 1. Bore Hole Shear, Earth Settlement, and Earth Penetrometer Probes

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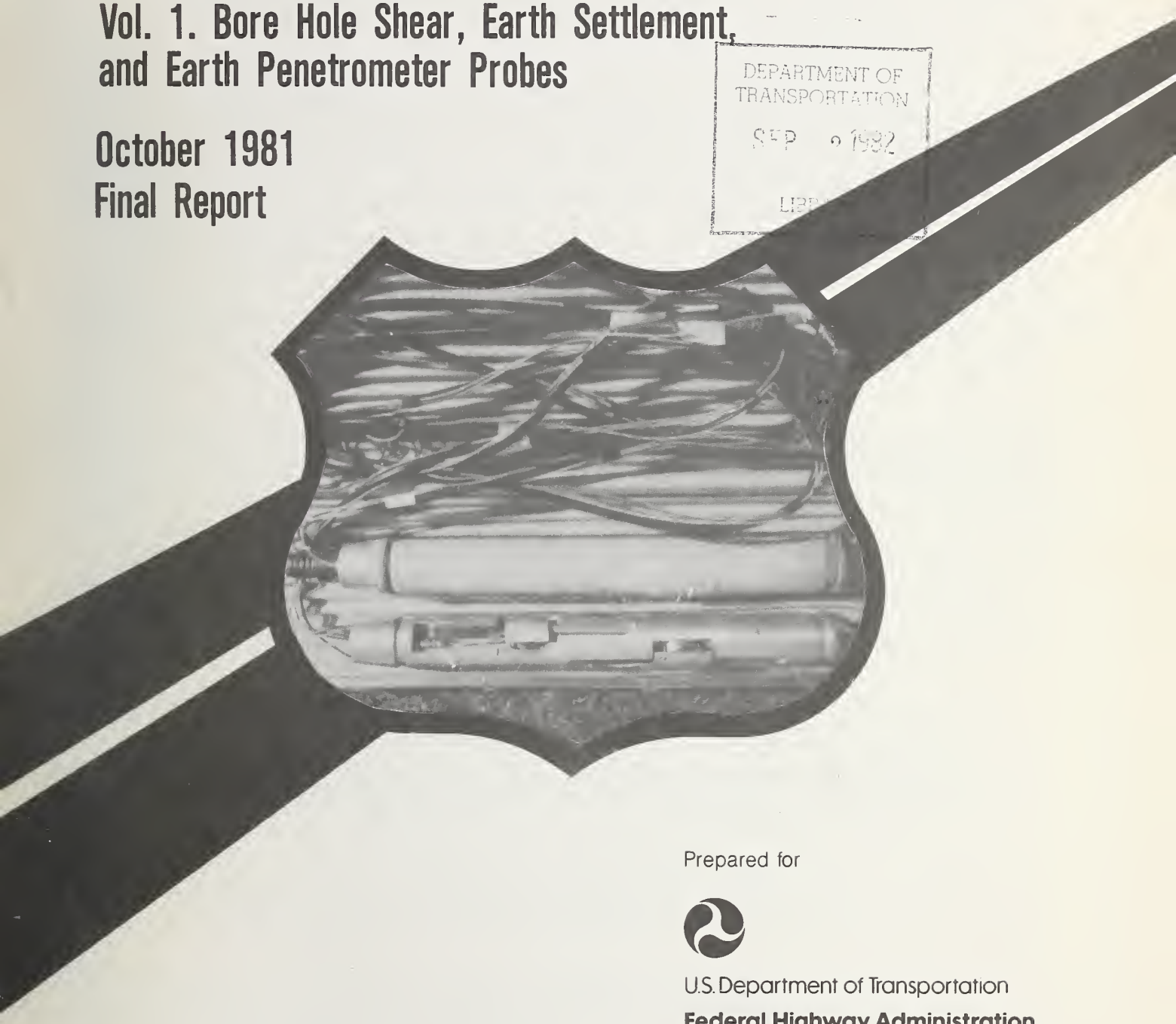

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FOREWORD

This report contains the results of a research effort conducted by the Federal Highway Administration through Soil Systems Incorporated to study three newly developed concepts for in situ testing and to evaluate their applicability to the design and construction of highway tunnels.

This is the first of the five volumes. Volume 2, FHWA/RD-81/110, Subtitle: "Pressuremeter;" Volume 3, FHWA/RD-81/111, Subtitle: "Vane Shear and Cone Piezometer;" Volume 4, FHWA/RD-81/112, Subtitle: "Static Penetrometers," Volume 5, FHWA/RD-81/113, Subtitle: "Dutch Cone Penetrometer;" will be published when they become available by the end of 1982.

This report presents the study of the three new in situ testing concepts: the Bore Hole Shear (BHS), the Bore Hole Earth Settlement Apparatus (BESA) and the Bore Hole Electronic Earth Penetrometer (BEEP). The BHS generates shear strength parameters, C and Ø within the soil walls of a bored hole. The BESA can be used in three distinct ways: as a similitude model load test; as an elastic modulus test; as an in situ consolidation test. The BEEP is a horizontal static penetrometer which reacts against the sides of a bore hole to provide penetration resistance data. Correlation testing for all three concepts has proven very encouraging.

This report should serve the needs of geotechnical, structural, and civil engineers who are planning, designing or thinking to construct an underground structure.

Copies of the report are being distributed by FHWA transmittal memorandum. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Charles F. Scheffey
Director, Office of Research
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16. Abstract

This research involves an analysis of three in situ testing methods for possible use in tunnel design in soft ground. These three in situ testing methods, the Bore Hole Shear (BHS), Bore Hole Earth Settlement (BESA) and Bore Hole Electronic Earth Penetrometer (BEEP) are in various stages of refinement. All show promise as practical methods of obtaining soil engineering data in a bored hole. The BHS is presently in commercial production. The BEEP requires only additional instrument research design to become manufacturable, and the BESA requires additional applied research and correlation testing prior to final design and manufacture.

This is the first of five volumes. Volume 2, FHWA/RD-81/110, Subtitle: "Pressuremeter;" Volume 3, FHWA/RD-81/111, Subtitle: "Vane Shear and Cone Piezometer;" Volume 4, FHWA/RD-81/112, Subtitle: "Static Penetrometers;" and Volume 5, FHWA/RD-81/113, Subtitle: "Dutch Cone Penetrometer;" will be published when they become available by the end of 1982.

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TABLE OF CONTENTS

	Page
BORE HOLE SHEAR DEVICE (BHS)	
Introduction	1
Synopsis	1
Uses in Engineering Design	2
Slope Stability Failures Including Landslides	2
Earth Anchors	3
Friction Pile Capacity	4
Foundation Design	4
Retaining Wall Stability	4
Soil Parameters Obtained	4
Methodology and Testing	6
Limitations	8
Limitations Imposed by Site Conditions	9
Soil Properties and Empirical Relationships	11
In-House Research and Development	13
Case Histories	16
No. 1 - Chapel of All Faiths	17
No. 2 - Chimney Hill Apartment Investigation	18
General Site Conditions	19
Subsurface Conditions	19
Shear Strength Evaluation	22
Slope Stability Analysis	22
Discussion	36
Recommendations	37
No. 3 - Bent Creek Apartments	38
Field Investigations	39
Laboratory Investigations	41
Site Conditions	41
Subsurface Conditions	42
Slope Stability	42
Reinforced Concrete Retaining Wall	43
Caisson Wall	43
Granite Masonry Gravity Retaining Wall	43
Reinforced Earth Gravity Retaining Wall	43
No. 4 - Rich's Computer Center Site, Georgia Tech Research	47
Bore Hole Number 1	48
Bore Hole Number 2	48
Bore Hole Number 3	52
No. 5 - Steam Plant Site, Pensacola, Florida	60
Site and Subsurface Conditions	60
References Cited	71
BORE HOLE EARTH SETTLEMENT APPARATUS (BESA)	
Introduction	74
Synopsis	74
Uses in Engineering Design	75

TABLE OF CONTENTS (Cont.)

	Page
Soil Parameters Obtained	75
Elastic Modulii	76
Methodology and Testing	76
Limitations Imposed by Site Conditions	83
Soil Properties and Empirical Relationships	83
In-House Research and Development	84
Case Histories	87
Plate Bearing Test	87
Spread Footings	92
Mat Foundation	99
Similitude Model Load Tests in Sand	103
Similitude Model Load Tests in Clay	115
References Cited	122

BORE HOLE ELECTRONIC EARTH PENETROMETER (BEEP)

Introduction	124
Synopsis	124
Uses in Engineering Design	125
Subsurface Investigation and Feasibility Studies	125
Foundation Design	126
Shallow Footing Inspection	127
Soil Parameters Obtained	128
Methodology and Testing	129
Accuracy	133
Limitations Imposed by Site Conditions	134
Soil Properties and Empirical Relationships	135
In-House Research and Development	136
Case Histories	142
No. 1 - Warehouse Structure Failure Investigation	142
No. 2 - Four-Story Office Building Foundation Investigation	145
No. 3 - Apartment Complex, Richland County, South Carolina	148
No. 4 - Education Facility Subsurface Investigation	151
No. 5 - Warehouse Facility Subsurface Investigation and Engineering Evaluation	155
References Cited	158

LIST OF FIGURES

		PAGE
1.	BHS Schematic and Sideview	5
2.	Boring Plan - Chimney Hill Project	21
3.	BHS Shear Envelope- Bore Hole #2, 2 Feet Deep	23
4.	BHS Shear Envelope- Bore Hole #1, 4½ Feet Deep	24
5.	BHS Shear Envelope- Bore Hole #3, 2 Feet Deep	25
6.	BHS Shear Envelope- Bore Hole #3, 4 Feet Deep	26
7.	BHS Shear Envelope- Bore Hole #4, 3½ Feet Deep	27
8.	BHS Shear Envelope- Bore Hole #4, 4 Feet Deep	28
9.	BHS Shear Envelope- Bore Hole #5, 2¼ Feet Deep	29
10.	BHS Shear Envelope- Bore Hole #6, 2¼ Feet Deep	30
11.	BHS Shear Envelope- Bore Hole #7, 2 Feet Deep	31
12.	BHS Shear Envelope- Bore Hole #7, 4 Feet Deep	32
13.	Generalized Section A-A, Slope Stability Analysis, Chimney Hill Project	33
14.	Generalized Section B-B, Slope Stability Analysis, Chimney Hill Project	34
15.	Boring Plan, Bent Creek Apartment Complex	40
16.	Proposed Reinforced Earth Retaining Wall, Bent Creek Project	45
17.	Section A-A (Typical) Reinforced Earth Retaining Wall, Bent Creek Apartment Project	46
18.	BHS Data, Rich's Computer Center, Bore Hole #1	49
19.	Triaxial Shear Test, Rich's Computer Center, Bore Hole #1	50
20.	BHS and Triaxial Data, Rich's Computer Center, Bore Hole #1	51
21.	BHS Data, Rich's Computer Center, Bore Hole #2	53
22.	Triaxial Shear Data, Rich's Computer Center, Bore Hole #2	54
23.	BHS and Triaxial Data, Rich's Computer Center, Bore Hole #2	55
24.	BHS Data, Rich's Computer Center, Bore Hole #3	56
25.	Triaxial Shear Test, Rich's Computer Center, Bore Hole #3	57
26.	General Soil Profile Before Dike (1967) Crist Steam Plant	61
27.	General Soil Profile After Dike Consolidation, Crist Steam Plant	63
28.	Moisture Content Versus Depth, Overburden Pressure Versus Depth, Crist Steam Plant	65
29.	Bore Hole Shear and Vane Shear Data, Crist Steam Plant	68
30.	BESA Apparatus	77
31.	Load Deflection Curve for Bore Hole Plate Load Test	88
32.	Plate Load Test Modulus Data	90

33. Modulus Data, Donaldson Air Force Base ---------- 94
34. Soil Test Boring, Donaldson Air Force Base ---------- 95
35. Modulus Data, Atlanta Internationale Hotel ---------- 101
36. Layers Used in Layer-Strain Method and Variation of σ_o -------- 102
37. BESA Similitude Tests, Sand ---------- 106
38. BESA Similitude Tests, Sand ---------- 107
39. BESA Similitude Tests, Sand ---------- 108
40. BESA Similitude Tests, Sand ---------- 109
41. BESA Similitude Tests, Sand ---------- 110
42. BESA Similitude Tests, Sand ---------- 111
43. 15" by 20" Prototype Plate Load Test, Sand ---------- 113
44. BESA Similitude Tests, Clay ---------- 118
45. BESA Similitude Tests, Clay ---------- 119
46. BESA Similitude Tests, Clay ---------- 120
47. BEEP Apparatus, 2nd Generation ---------- 130
48. BEEP Nomograph, 2nd Generation Unit ---------- 132
49. BEEP Single Test Penetration Resistance vs. Dynamic Penetrometer ("A & P") ---------- 138
50. BEEP Average Penetration Resistance vs. Dynamic Penetrometer ("A & P") ---------- 139
51. BEEP data vs. SPT and "A & P" data ---------- 140
52. Boring Plan - Apartment Complex, Richland County South Carolina ---------- 150
53. Boring Plan - Continuing Education Building, Columbus,Georgia- 153
54. Boring Plan - Shaheen Warehouse, DeKalb County, Georgia ------ 157

LIST OF TABLES

		PAGE
1.	Comparative Data - Bore Hole Shear Device	15
2.	Summary of Slope Stability Analysis Results	35
3.	BHS and Triaxial Data, Rich's Computer Center	59
4.	Unit Weight and Moisture Content Data, Crist Steam Plant Site-	64
5.	In-Situ Test Results, Crist Steam Plant Site	69
6.	Plate Load Test Data and Calculations using BESA Elastic Modulus Data and Layer Strain Analysis	91
7.	Settlement Calculations Comparing BESA Modulus Data, Triaxial Modulus Data, and Measured Settlements,Donaldson Center Mat Load Test	96
8.	Settlement Calculations Comparing BESA Modulus Data, Triaxial Modulus Data, and Measured Settlements,Donaldson Center Hangar Footings	97
9.	Soil Test Data	105
10.	BESA Similitude Data, Sand Site	114
11.	BESA Similitude Data, Clay Site	117
12.	BEEP Penetration Resistances	146
13.	BEEP Penetration Resistances	149
14.	BEEP Penetration Resistances	154
15.	BEEP Penetration Resistances	156

Introduction

The Bore Hole Shear Device (BHS) also called the Iowa Bore Hole Shear Device is a commercially available testing instrument and method which has had considerable use during the past ten years by both governmental agencies and geotechnical consulting organizations. The instrument and test method is capable of determining the shear strength parameters, C and Ø, in most soil types. It is roughly comparable in concept, to an inside-out direct shear test, performed in-situ on the sides of a bored hole.

Synopsis

The Bore Hole Shear Device (BHS) study discusses actual uses of the BHS in solving engineering problems. Landslide failure analysis, earth anchor design, friction pile design, and foundation design uses are discussed. The difficulty of comparing data with more conventional shear strength testing methods such as in the triaxial test, is also discussed. The main limitation to use of the present commercially available instrument is in mobilizing the cohesion in moderately cohesive to highly cohesive soils. The rapidity and ease of testing is described, and the positive aspects of a reliable method of obtaining the shear parameters, C and Ø in-situ is shown.

USES IN ENGINEERING DESIGN

The purpose of the Bore Hole Shear Device (BHS) test is to determine shear strength parameters in-situ. Existing slopes, whether they consist of natural soils or fill materials, can be tested with this method. The test itself approaches either a drained or an undrained test depending upon the type of soil being tested and the procedures used in performing the test. This factor must be considered in evaluating test results and in order to properly utilize the shear strength parameters obtained for slope evaluation and design. Because most earth dam embankment slopes are designed to be constructed of fill materials, the Bore Hole Shear Device would often not be applicable for embankment analyses during the planning and design phases of a project. On the other hand, the foundation soils supporting the dam many be evaluated with the BHS test as well as other subsurface data. There would also be nothing to preclude performance of BHS tests on compacted fills in a test field section.

Slope Stability Failures Including Landslides

Perhaps the most prevalent use of the BHS to date is in evaluation of existing slopes with impending or active landslides. The general nature of the BHS test, i.e., the in-situ evaluation of soil shear strength, is particularly well suited to testing and evaluation of slope failure problems. The BHS method has been used on landslides or slope failures ranging in size from very small surficial slope displacements to relatively massive landslides with elevation differences in excess of 30.5 meters (100 feet) from base to top of slope. In many cases, thin wall tube samples are obtained and the BHS tests are performed in the holes created by the tube. In conjunction with BHS testing, laboratory triaxial tests may be performed and often are performed with the undisturbed samples obtained.

The BHS test has been used to evaluate the effectiveness of chemical stabilization of an active landslide.[1] Shear strength parameters were determined by the BHS prior to effecting chemical stabilization. Subsequent to chemical stabilization, new shear strength data were obtained. This process aided in evaluating the effectiveness and long term potential stability of the chemically stabilized slope.

Comparisons between shear strength parameters obtained from BHS tests and laboratory triaxial tests are generally close, with the exception that moderately to highly cohesive soils often prevent full mobilization of the cohesive shear strength parameter in the BHS tests.[2] This will be further discussed in a later section of this study.

Advantages found in the use of bore hole shear testing for landslide evaluation include the following:

(1) rapidity in obtaining shear strength parameters;
(2) portability of equipment and therefore ability to obtain data in relatively inaccessible locations on a slope;
(3) ability of the test method to obtain data in relatively thin layers of very sensitive and weak soils within which undisturbed samples may be impossible to obtain;
(4) ability to obtain more information for a fixed cost because of the relative speed at which the tests are taken;
(5) ability to obtain shear strength data within the actual zone of soil undergoing failure.

Earth Anchors: The BHS test is used to provide the shear strength parameters needed to calculate uplift capacity of earth anchors. Actual uses include pin piles, caissons and drilled piers, and other friction pile anchors including auger cast piles which are designed to provide uplift and/or pulling capacity. Examples of problems solved with the assistance of BHS data include resistance from buoyant forces in swimming pools; uplift resistance of foundations required because of eccentric and/or wind loadings that produce uplift forces; and anchors used in bracing and retaining wall design.

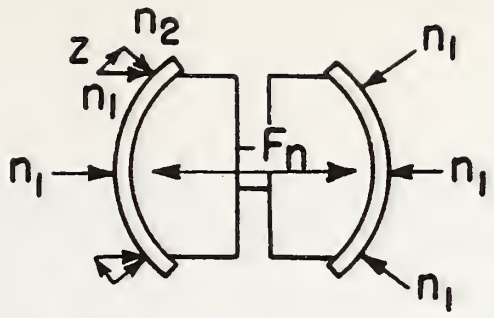
Friction Pile Capacity: The BHS has been used a number of times to determine the vertical downward friction capacity of different types of friction piles and caissons (drilled Piers). An important determination in evaluating this test method for that particular purpose is to determine whether shear will occur between the pile surface and the soil and therefore constitute a soil-to-pile failure, or whether it will occur within the soil itself. In general, concrete piles and concrete drilled piers are found to be sufficiently rough to result in a soil-to-soil shear failure. On the other hand, "H" piles and driven steel pipe piles or step-tapered piles may fail in a soil-to pile shear mode. In the latter case, use of bore hole shear data may not be accurate and may in fact be unsafe.

Foundation Design: The BHS method has been successfully used in determining shear strength parameters for calculating foundation capacity in very weak soils for construction of a dike system. In general, the BHS can be used in any type of foundation investigation to determine bearing capacity criteria for design. A somewhat similar example of BHS data being used for foundation design is in the evaluation of pile cap capacity to resist horizontal forces. Soil shear strength parameters are needed in order to calculate passive earth pressure resistance.

Retaining Wall Stability: The BHS has been used to provide shear strength parameters in order to calculate earth pressure data with which to design retaining walls, basement walls, and other structures which retain soil.

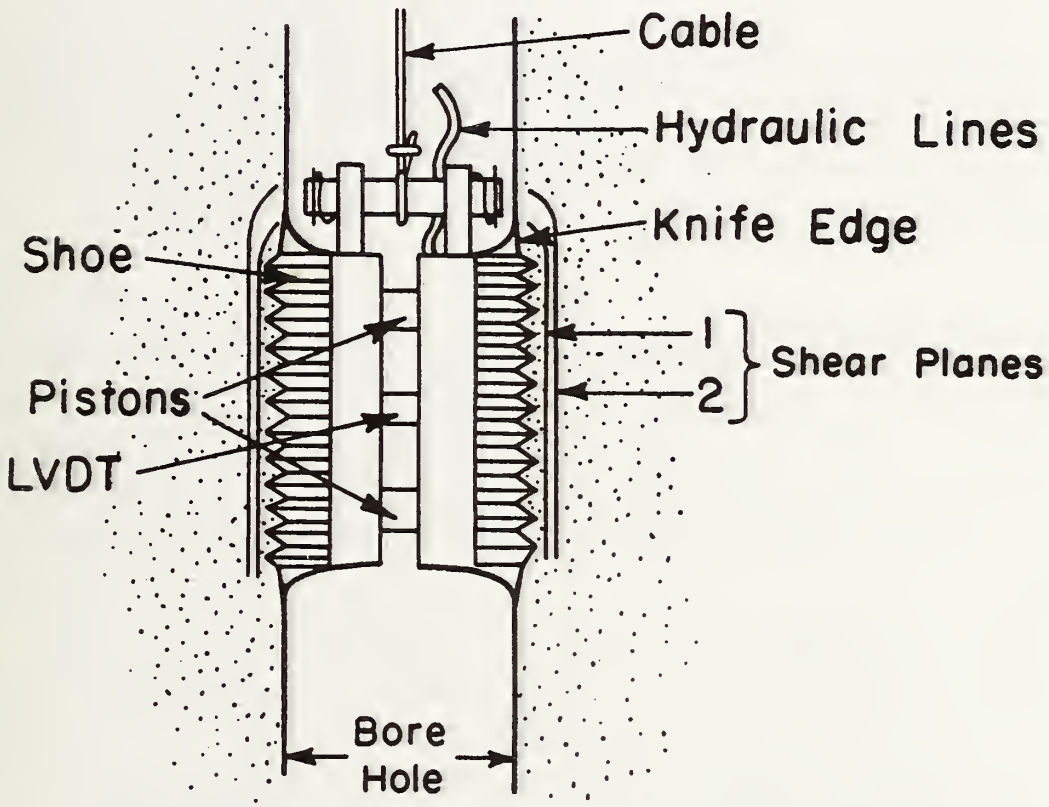
Soil Parameters Obtained

The Bore Hole Shear Device results in the separable determination of the two soil shear strength parameters - cohesion (C) and internal friction (ϕ). The test is essentially a direct shear test performed on soil at the sides of a bored hole. The method of operation involves the application of a predetermined pressure normal to the incipient shear plane, and controlled application of a measurable shearing stress to cause failure. Shear planes are developed within the soil generally as shown in Figure 1. The test may be essentially



Schematic of soil bore-hole shear device.

(a) Top view.
Reduction of circumferential shearing resistance Z should give more even distribution of normal pressures n_1.



(b) Side view.
Successive shear planes 1 and 2 may relocate outward as soil compacts close to the apparatus.

Figure 1 - BHS Schematic and Sideview

a drained test or an undrained test depending upon the soil type being tested and the specific procedures being utilized in the performance of the tests. Generally, for granular, relatively free draining soils, the test represents a consolidated drained test. For relatively impervious soils, the test is generally a consolidated undrained test. For soils within intermediate permeabilities, the test can usually be considered a consolidated, partially drained test. As pointed out by Schmertman[3], the BHS test is essentially a stage test in that shearing is induced repeatedly in the same soil mass, at successively higher normal stresses.

A supplemental adaption to the BHS device allows for evaluating creep resistance of soils. This test modification is rather rare, and not presently in general use. Stress vs. strain data are also available from the BHS test performed to obtain C and ϕ. However, the use of stress-strain data for engineering design is presently limited.

Methodology and Testing

The shear strength parameters, C and ϕ, are obtained by lowering the cylindrical device into a predrilled bore hole, expanding the two halves of the split cylinder with a preselected and regulated contact pressure (normal stress) and pulling vertically upward at a controlled rate of strain while measuring the maximum pulling force mobilized to cause yielding or failure.

The test procedure can be outlined as follows:

(1) Lower BHS to desired testing elevation in a predrilled bore hole of controlled diameter.

(2) Expand the split halves of the BHS to a predetermined gage pressure corresponding to a desired initial normal stress.

(3) Allow a predetermined time to elapse for consolidation of the stressed soil.

(4) Apply vertical stress by pulling a rod attached to the BHS. A mechanical handle-gear pulling apparatus is used. Rate of strain can be relatively well controlled.

(5) Vertical stress is measured by load cells incorporated in

the pulling mechanism. The maximum gage reading recorded is converted to vertical stress through a conversion table or chart.

(6) Using the Mohr coordinate system, the operator plots shear strength vs. normal stress and obtains one point from steps 1 through 5 above.

(7) The vertical stress applied in step 6 is removed after a maximum reading has been taken. The normal stress (step 2) is then increased to a predetermined value, and steps 2 through 6 are repeated. This yields a second point on the Mohr envelope.

(8) Additional points are obtained as needed, and as possible within the particular soil tested. In some cases the BHS may have to be moved vertically 15 or 20 centimeters (6 or 8 inches) in order to provide additional point test results.

The points obtained nearly always plot as a straight line. Exceptions include the first point, which is often low (due to incomplete seating of the corrugated plates), and points at higher pressures which may be affected by pore-water pressure. This can be verified by increasing the consolidation prior to shearing the next point in order to increase the percent drainage and approximate a drained test. The slope of the linear failure envelope corresponds to the soil internal friction angle, ϕ, and the intercept is the cohesion C.

These parameters, C and ϕ, have been successfully obtained with the BHS method in a wide variety of soil types and geological provinces, mainly in the midwestern and southeastern United States. The Bore Hole Shear Device is known to have been successfully used in the states of Iowa, Kansas, Georgia, Florida, North Carolina, South Carolina and Hawaii, and in the following soil types: soft to very stiff sandy micaceous silt (residual soils); loose to dense silty sand (residual soils); loose to firm uniform graded sand (aolian soils); soft to firm silty clays (alluvial soils); soft to stiff clayey silts (alluvial soils); poorly compacted to well compacted silty sands, sandy silts, clayey silts

(fill soil); silty clays (glacial till); silts and clays (tropical soils); clayey silts (loess); and clayey sands and sandy clays (coastal plain soils).

Making a fully acceptable comparison of BHS accuracy with the "standard" of triaxial data is difficult at best. Problems in making absolute comparisons include the following factors:

(1) difference in sample disturbance;
(2) predetermined direction of shear in the BHS;
(3) drainage differences accompanying either test;
(4) known inaccuracies in the triaxial test; and
(5) testing of different soils (this is minimized when the BHS test is performed on soil walls in a hole made by a thin walled tube, while triaxial tests are performed on samples from the same tube).

Limitations

In general, the greatest limitation to accuracy in the BHS is in mobilizing the cohesion in a moderately to highly cohesive soil.[2] A rule of thumb developed from a significant number of test comparisons is that if the cohesion parameter of a soil in the triaxial (undrained) test is greater than 55.1 kN/m^2 (8 psi) then the BHS is likely to be unable to obtain the full cohesion. CH, MH and some CL soils may be subject to this overly conservative test characteristic with the BHS. Sands, silts, and low cohesion clays can normally be tested by the BHS method with reasonably accurate results.

The best indicator of data accuracy for BHS testing may well be the testing of active landslides. Since the factor of safety is known to be approximately 1.0 in an active slope failure, and accepted theories of stability evaluation involve the C and ϕ parameters as variables, an examination of computed factors of safety using actual Bore Hole shear data in areas of active sliding can constitute a valid appraisal of BHS accuracy. Several slope stability studies utilizing BHS data are discussed in the Case Histories section of this study. Dozens of landslide investigations have been performed with data provided by the BHS.

Factors of safety in areas of active landslides where the factor of safety is known to be 1.0 usually compute with BHS data to between 0.95 and 1.10.

Another indicator of accuracy is comparison of BHS data with the generally recognized "standard" in the accepted state of the art, which is the triaxial shear test. BHS data generally compare well with triaxial data and usually plot intermediate between consolidated drained and consolidated undrained tests, depending upon soil type tested.

Limitations Imposed by Site Conditions

The Bore Hole Shear Test is dependent upon the bore hole remaining open, as is the case for any in-situ testing where recovery of the tools and equipment is necessary. Cave-ins and sloughing of soil (squeezing-in) which can occur in certain soils may hinder or prevent testing. At times, squeezing-in of soils during successful tests results in the device becoming temporarily stuck in the hole. However, no cases are known by the author wherein the device was lost in a hole due to cave-ins or sloughing. Successful methods of reducing the cave-in and sloughing occurrences in sensitive soils include: use of the cutter tool supplied with the equipment kit; casing the hole to the depth immediately above the depth tested; use of the drilling mud; and testing the soil immediately after the hole has been drilled.

The Bore Hole Shear Test method requires that a relatively smooth hole of a certain size limitation be drilled. For the commercial model, the hole should be between 7.6 centimeters (3.0 inches) and 8.8 centimeters (3.5 inches) in diameter. A limitation imposed by site conditions includes wallowing of drilling tools resulting in holes being too large or too irregular to test.

One successful method to avoid wallowing of drill tools or other problems related to developing a relatively smooth hole of the correct diameter is to utilize 7.6 centimeter (3-inch) thin wall tubes to both obtain a sample and to provide a correct size hole for testing. Mechanical drill can be accomplished to a depth just above test depth(s). A thin walled tube can subsequently be advanced to produce a correct hole. The sample obtained may be tested in the laboratory to produce additional data and possible research correlation. However, in most cases a proper

sized drill bit or auger and careful drilling techniques are adequate. Hand drilling to reasonable depths (normally 0.3 to 3 meters or 1 to 10 feet) has proven practical under most conditions.

Mudded holes have successfully been tested; however in certain soils the drilling mud can be expected to cause a change in the engineering properties of the tested soil zone. The author has found that drilling mud in sands, sandy silts, and silty sands generally does not prevent accurate tests from being performed. When a poor test does result due to the effect of drilling mud within the sheared zone, the questionable test results are normally detected by the unreasonably low shear strengths and friction angles which result. These inaccurate tests are then discarded.

Another limitation, at least with the presently available commercial model of the BHS, is in shear plates not becoming fully seated in hard or dense soils. Potentially, the problem is capable of being solved through a combination of redesign of shear plates and the ability to provide increased normal pressure ranges. In fact, a modified version of the BHS has been successfully designed and tested to determine shear strengths of coal.[4]

With the presently available commercial BHS model, there is a definite limit to the consistency of soils which may be successfully tested. The limitation appears more severe in clayey soils than non-cohesive soils. Although data are not sufficient to accurately define limits, a rough estimate of the maximum soil consistency which should be within a testable range for the BHS is shown below:

Non-cohesive soil - 100 blows per foot standard penetration test

Cohesive soil - 30 blows per foot standard penetration test

A more definitive measure of soil engineering properties is in the cohesive shear strength which can be successfully tested with available commercial equipment and standard shear plates. Presently the maximum reliably testable cohesion with standard plates is on the order of C = 55.1 kN/m^2 (8 psi)[2]. This limitation is being changed by the development of smaller plates specifically designed for very stiff or hard soils (2). These plates are easily attachable in place of the

standard shear plates. Results of recent testing and research at Iowa State University indicate considerable promise with the special plates.

Another limitation which has occured in weak soils, particularly with very loose sands, is a bearing capacity type of failure caused by the normal stress applied to the shear plates. However, if the normal stresses are kept low, at least one or two point tests can usually be performed even in a very loose sand, before bearing capacity cave-in occurs. Additional test points can be determined by retracting the plates and moving the device vertically at least 15 centimeters (6 inches). It should also be mentioned that in potential cave-in or bearing capacity problem soils (which are usually cohesionless sands), even a single point test can provide valuable shear strength information since the cohesive parameter for a cohesionless sand can be assumed to be zero.

One caution for accurate results is not to perform tests at different elevations within a drilled hole without removing the device and cleaning the plates. Soil may adhere to the surface of the plates and affect tests at a different soil stratum. However, the cleaning process can normally be accomplished with wire brush and bucket in several minutes.

Although not actually a limitation, the relative ease and speed of performing a test could potentially lead to a problem. A technician without an adequate soil engineering education and experience, could be easily trained to perform the test. However, proper operation and interpretation of data while a test is in progress is essential in order to accurately obtain the in-situ shear strength with the BHS concept.

Soil Properties and Empirical Relationships

The primary paramters obtained by the BHS test are the soil shear strength parameters C and Ø. In general, all soil mechanics problems utilizing c and Ø can use data obtained from this test for design, unless required draining conditions during the test are different from those obtained from the BHS test. The design soil engineer must realize the characteristics of the test method and data obtained in order to evaluate the applicability of the BHS data. One factor which should be considered in applying these data to design problems is the drainage

condition of the soil being tested, i.e., whether the test is a drained shear test, an undrained test, or somewhere in between. Depending on soil type and method of testing with the BHS, any one of these three conditions can prevail. An experienced soil engineer can determine appropriate drainage conditions for a given situation.

Another characteristic of the BHS test that should be evaluated by an experienced soil engineer in utilizing data for design is in the predetermined direction of shear. If the soil is a varved clay or otherwise stratified soil, shear strength parameters obtained in a vertical soil may not accurately represent the weakest shear plane or the mass shear strength.

Relationships used in design include design problems listed below:

(1) Earth Dam and Embankment Slope Design: BHS data for design of safe slopes have utilized various slope stability formulas to include the Simplified Bishop method, and the Janbu et al (1956) method. Both hand-calculated as well as computer solutions to slope stability evaluations have utilized BHS data with reasonable success.

For example, the formula $$F = \frac{\sum_{i=1}^{i=n} [\bar{c}\Delta x_i + (W_i - u_i \Delta x_i)\tan \bar{\phi}][1/M_i(\theta)]}{\sum_{i=1}^{i=n} W_i \sin \theta_i}$$

where $M_i(\theta) = \cos \theta_i (1 + \frac{\tan \theta_i \tan \bar{\phi}}{F})$ represents one trial solution to determine a factor of safety utilizing the Simplified Bishop method of slope stability analysis.[5] The $\bar{C}$ and $\bar{\phi}$ for this solution may be provided by the BHS data as obtained directly, or, depending upon the assumptions made and the stability situation being investigated (sudden drawdown condition, overtopping condition, etc.), the BHS data could possibly be modified to approximate a different drainage condition.

(2) Earth Pressures: BHS data have been used to determine earth pressures for retaining wall design. For example, using the equation[6]

$$E_A = \frac{\gamma H^2}{2} \tan^2 (45° - \frac{\phi}{2}) - 2cH \tan(45° - \frac{\phi}{2})$$

where E_A is the active earth pressure, and the C and ϕ shear strength parameters can be supplied by BHS data.

(3) Anchors: Earth anchor design has been accomplished with BHS data. Examples have been cylindrical anchors where a relationship between contact area and shear strength has been utilized for uplift resistance calculations. A simplified equation would be $F = [C + N \tan \phi] A$, for vertical pull, where A = perimeter area of the cylindrical anchor, or πdh. N will vary with depth and lateral earth pressure. For deadman anchorages, the equations $E_p = \frac{\gamma H^2}{2} K_p$ where $K_p = \tan^2 (45° + \frac{\phi}{2})$ for sands and $E_p = \frac{\gamma H^2}{2} + 2CH$ for plastic clay soils can be used. Again, C and ϕ, the shear strength parameters, can be provided by the BHS.

(4) Landslides: Numerous active landslides have been investigated utilizing data obtained from BHS tests. In fact, this test is unique in being able to obtain C and ϕ information separately within the failing soil zone. With the BHS data obtained, one can calculate the existing factor of safety, using accepted theories and reasonable assumptions . An active landslide is known to have a factor of safety of 1.0 by definition. Calculations of active slides with data obtained can verify the mechanism of failure and the validity of assumptions made. General methods of analysis which have successfully been used are failure wedge, circular arc, and non-circular Janbu methods.

In-House Research and Development

"In-house" research in the use of the Iowa Bore Hole Shear Device is considered to be that research conducted under the supervision of the author. This summary includes two major efforts, the first in conjunction with Iowa State University in the performance of a

Masters thesis, and the second, performed by a graduate student of the Georgia Institute of Technology.

In the original research performed at Iowa State University, four soils were tested for initial comparison of field and laboratory data. These soils included a stable dune sand, an alluvial loam, a heavy textured clay, and a silt loess. All were geologically recent deposits with little or no history of preconsolidation. The tests were conducted at depths ranging from 10 centimeters (4 inches) to 3 meters (10 feet). Table 1 gives comparison data from Bore Hole Shear Tests (BHS), direct shear tests (D.S.), and triaxial (T) tests on the four soils. Table 1 also includes statistical information concerning the accuracy of some of the data obtained. For the soil property ranges tested, the Bore Hole Shear Test was reasonably reliable and required considerably less time to perform than laboratory direct shear or triaxial tests. In some cases, the Bore Hole Shear Test proved potentially superior to laboratory testing since considerably more tests could be performed in the same amount of time or for the same cost, allowing a statistical treatment of data.

The work performed at the Georgia Institute of Technology included testing of Piedmont residual soils. Two locations were chosen for field testing. Site number one was chosen because a commercial testing laboratory had previously performed soil test borings at this site with standard penetration tests. Also, thin walled tube undisturbed samples had been taken in intermediate zone micaceous silts and were available to be tested in a laboratory triaxial test apparatus. Borings were hand augered a lateral distance of approximately one foot from original soil test borings to increase the assurance that the tests would be performed in soils similar to those taken in the undisturbed samples. A split spoon sampler was attached to a drill rod and hand driven into the bottom of the holes. Samples were removed and determinations of dip and strike were made. The dip was determined to be at an angle of 45 degrees to the north. Accordingly, each test was planned to be made approximately one foot lower than the average depth of the undisturbed sample. The accuracy of the original test

TABLE 1

Comparative Data - Bore hole Shear Device

Soil Type	Ø, Degrees			C, psi		
	Bore Hole Shear	Direct Shear	Triaxial	Bore Hole Shear	Direct Shear	Triaxial
Dune Sand	36.9 ± .5	36.5 ± .3	35.8[b]	0.4 ± .3	0.3 ± .3	0.3[b]
Sandy Loam	44.8	44.0[c]	---	1.2	0.8[c]	----
Clay	5.0 ± 2.6	----	2.5 ± 2.3[c]	10.8 ± 1.0	-----	9.0 ± 2.3[c]
Loess (silt)	25.4	24.1	28.9	1.0	1.3	2.4

a The ± entry indicates one standard deviation from the mean

b Only one test performed

c Only one test series performed

Note: To convert psi to kN/m^2, multiply by 6.89

depths is subject to question since the driller's accuracy of depth measurement was probably only within one foot. Results of field Bore Hole Shear testing and laboratory quick triaxial tests are presented in the case study segment of this report.

It should be pointed out that the research and development performed both at Iowa State University and the Georgia Institute of Technology utilized the third generation hydraulic Bore Hole Shear unit which was a relatively crude instrument. Subsequent to this research and development, the Bore Hole Shear Device was redesigned for commercial use. The redesigned unit is a precision instrument utilizing pneumatic pressure rather than hydraulic. Although the research and development performed by the senior author under the direction of Dr. R. L. Handy in developing the Bore Hole Shear Test device and concept was promising and showed significantly accurate correlation for use in design, the redesigned commercial model is even more accurate and reliable under most field conditions. This improved commercial model was utilized in four of the five case histories which will be presented in this study.

CASE HISTORIES

Five case histories have been selected to demonstrate the use of the BHS device to develop design data and soil properties. The first, the "Chapel of All Faiths" investigation, is a subsurface study in which overturning of a proposed 27.4 meter (90 foot) high bell tower was investigated. Shear strength parameters were needed by the structural engineer in analyzing resistance to overturning. The second two histories, the Chimney Hill Apartments and Bent Creek Apartments represent landslide investigations. The fourth case history - the Rich's computer center site - represent more basic testing to determine shear strength parameters for possible use in foundation and/or retaining wall design. The fifth case history, the Crist Steam Plant dike investigation, included a deformation investigation of soft clay and peaty soils.

Case History No.1: Chapel of All Faiths

In December, 1972, a subsurface investigation was performed under the supervision of the author, of a proposed chapel site in Tifton, Georgia.(7) Tifton is within the Coastal Plain Geologic Province in Georgia, and the site soils investigated were of depositional (sedimentary) origin.

Below a veneer of vegetation and topsoil averaging approximately 6 inches thick, the borings encountered a thin zone of firm clayey sands averaging approximately 1.2 meters (4 feet) thick. Below this clayey sand zone was a stratum of still to hard sandy clays which extended to boring termination depths. Results of a liquid limit test and a plastic limit test on these sandy clays indicated that the soils were of low plasticity, and not potentially susceptible to undergoing detrimental volume charges from future changes in moisture content.

Ground water was encountered at a depth of approximately 2.2 meters (7 feet) in each of the soil test borings within several hours after drilling.

The 12.2 meter (40 foot) by 33.5 meter (110 foot) chapel was to be of masonry construction with continuous(wall) footing foundations. Shear strength data would not normally be required, since standard penetration resistance information normally is adequate for foundation design. However, a bell tower was planned to be constructed approximately 12.2 meters (40 feet) from the chapel. The bell tower had interior dimensions of 1.2 meters (4 feet) by 2.4 meters (8 feet). Plans to resist overturning consisted of a combination of a drilled pier (caisson) and a dead man screw anchor system. The tower dead load was estimated to be 178 kN (40 kips).

Within the area of the proposed tower one soil test boring was drilled to a depth of 12.4 meters (40 feet. Bore Hole Electronic Earth Penetration (BEEP) tests were performed at five foot depth intervals. Bore Hole Shear Tests were performed at two depth locations - 1.5 meters (4.8 feet) and 2.3 meters (7.5 feet) within a sandy clay soil. Shear strength parameters obtained were as follows:

1.5 meter (4.8 foot) depth: $C = 430$ psf; $\phi = 33.5$ degrees
2.3 meter (7.5 foot) depth: $C = 0$ $\phi = 35.5$ degrees

No laboratory shear strength tests were performed because of budget limitations and the confidence the author had in the accuracy of the in situ bore hole shear test data obtained. For calculations of overturning and design of the earth anchor system, we recommend

using shear strength parameters of C = 0 and ϕ = 33.5 degrees. Actual design of the anchorage system was accomplished by the structural engineer, who used these values. The factor of safety used in that design, is not known.

Within the chapel foundation area, we recommended that the foundations be shallow wall and spread footings designed for a maximum allowable bearing pressure of 3000 psf. Similarly, for footings bearing on well engineered and controlled structural fill, we recommended an allowable bearing pressure for design of 3000 psf.

For the tower foundation design, because of uplift, we recommended the use of drilled and belled soil caissons designed for end bearing of 4000 psf for end bearing depths of 6 to 10 feet, and 5000 psf for end bearing depths from 10 to 40 feet.

Because of the relatively high ground water levels, we recommended minimizing caisson depths. However, it was pointed out that uplife pressures from the differential water levels could soften bearing soils, and that this probability increased with increased caisson depth below the water table. We further recommended casing the drilled holes with a steel casing and dewatering the excavations with a pump lowered to the bottom of the drilled holes. This method of dewatering, it was cautioned, may not prove adequate to prevent softening of the bearing soil, particularly if the drilled caisson were in excess of 10 feet below the water table.

The report was sent to the architect. Shear strength parameters were used by the structural engineer in design of the anchor system.

Case History No. 2: Chimney Hill Apartment Investigation

In September and October 1974, a study was made under the supervision of the author, with the assistance of Dr. Richard D. Bardsdale, Professor at the Georgia Institute of Technology.[8] This study was of a slope failure which had occurred adjacent to an apartment complex in Cobb County Georgia in the Atlanta metropolitan area. The purpose of the study was to determine the probably cause of the landslide and to make appropriate engineering recommendations to correct the problem. A summary of this study is presented below. This summary is taken from the engineering report.

General Site Conditions: A landslide developed on the north side of Building 2813 on an existing embankment slope. A tension crack approximately 1.5 meters (5 feet) long was observed on August 30, 1974. By August 31, the length of the tension crack had increased to 2.1 meters (7 feet) and the width of the crack had increased by a noticeable amount. The landslide then developed sometime between August 31 and September 3 when it was first noticed.

The landslide was slightly egg-shaped in plan view and was approximately 7.6 meters (25 feet) in width and 8.5 meters (28 feet) in length at the furthest points across. The landslide was located adjacent to the central part of the apartment building with the face of the slide being approximately 2.1 meters (7 feet) from balcony piers. To the northeast of the slide area and away from the apartment building, a gulley ran in a northeastern direction eventually leading down to the creek below. At the time of the field inspection on September 3, and subsequent inspection on September 4, a significant quantity of water was observed to be flowing out from the lower portion of the landslide zone. This area of the landslide was also observed at the time to be very soft and wet.

At the location of the landslide the ground dropped downward approximately 1 meter (3 feet) from the balcony piers. At this location, the embankment sloped downward at a gradient of 2.3 (H) to 1(V) a vertical distance of approximately 4.6 meters (15 feet) and then transitioned into a relative gentle slope or two dropped down into the drainage gulley.

At a location approximately 6.1 meters (20 feet) to the east of the slide area and 2.1 meters (7 feet) to the west of the existing wall, the slope increased downward approximately 1 meter (3 feet) from the edge of the building. At this location the embankment dropped at an average grade of approximately 1.8(H) to 1.0(V) for a vertical distance approximately 8.2 meters (27 feet).

Subsurface Conditions: The subsurface conditions in the slope behind Building 2813 was investigated by means of 13 hand auger test borings varying from 2.1 to 4.0 meters (7 to 13 feet) in depth. Because of rock fragments in the fill material hand augering was very difficult and quite often several offset holes had to be put down in order to complete the test hole. In the hand auger test holes the consistency of the soil was measured by means of a dynamic cone penetrometer. This

test consists of driving a 3.8 centimeter (1.5 inch) diameter cone by means of 66.8 N (15 pound) hammer falling 50.8 centimeters (20 inches). The soil consistency was the number of blows required to advance the cone 3.8 centimeters (1.5 inches). The soil consistency was generally determined at an interval of 0.3 meters (1 foot).

In addition to the hand auger test holes, 3 test holes were augered in the vicinity of the toe of the landslide by means of a CME-45 power auger mounted on an all terrain vehicle. Because of the relative steep angle of inclination, the rig could not be used on the side slopes. These test holes varied in depth from 5.2 to 7.0 meters (17 to 23 feet). The soil consistency was evaluated by means of the standard penetration resistance as defined by ASTM test method D 1586-67.

A plan view showing the location of both the hand auger and the mechanical auger borings is shown on figure 2.

The slope in the vicinity of the landslide was found in general to be underlain by very soft to soft brown sandy slightly micaceous silts. However, very soft clayey silts and rock fragments were also encountered in some portions of the fill. The depth from the surface to stiff to hard residual soil was found to vary from 3.0 to 5.2 meters (10 to 17 feet). The subsurface conditions at the top of the slope were found to be generally better than on the side slopes. The soil beneath the top portion of the slope consisted typically of soft to firm brown and red-brown sandy slightly micaceous silts. At approximately the location of the first interior column line of the structure, firm soils were encountered down to a depth of 2.1 meters (7 feet) where hand auger refusal was encountered. At the time the test holes were placed, the water table at the toe of the slope was approximately 0.7 meters (2.2 feet) below the surface.

The subsurface condition on the steeper slope located approximately 6.1 meters (20 feet) to the ease of the landslide and immediately west of the retaining wall were found to be somewhat poorer than those in the immedate vicinity of the landslide. A 1 to 3 meter (3 to 10 foot) thick layer of very soft to soft, brown and red brown sandy and slightly clayey silts was encountered at this location. Beneath these softer upper soils, a layer of firm brown sandy slightly micaceous silt was encountered varying in thickness from 0 to 1.5 meters (0 to 5 feet). Hand auger refusal in residual soil was encountered at depths varying

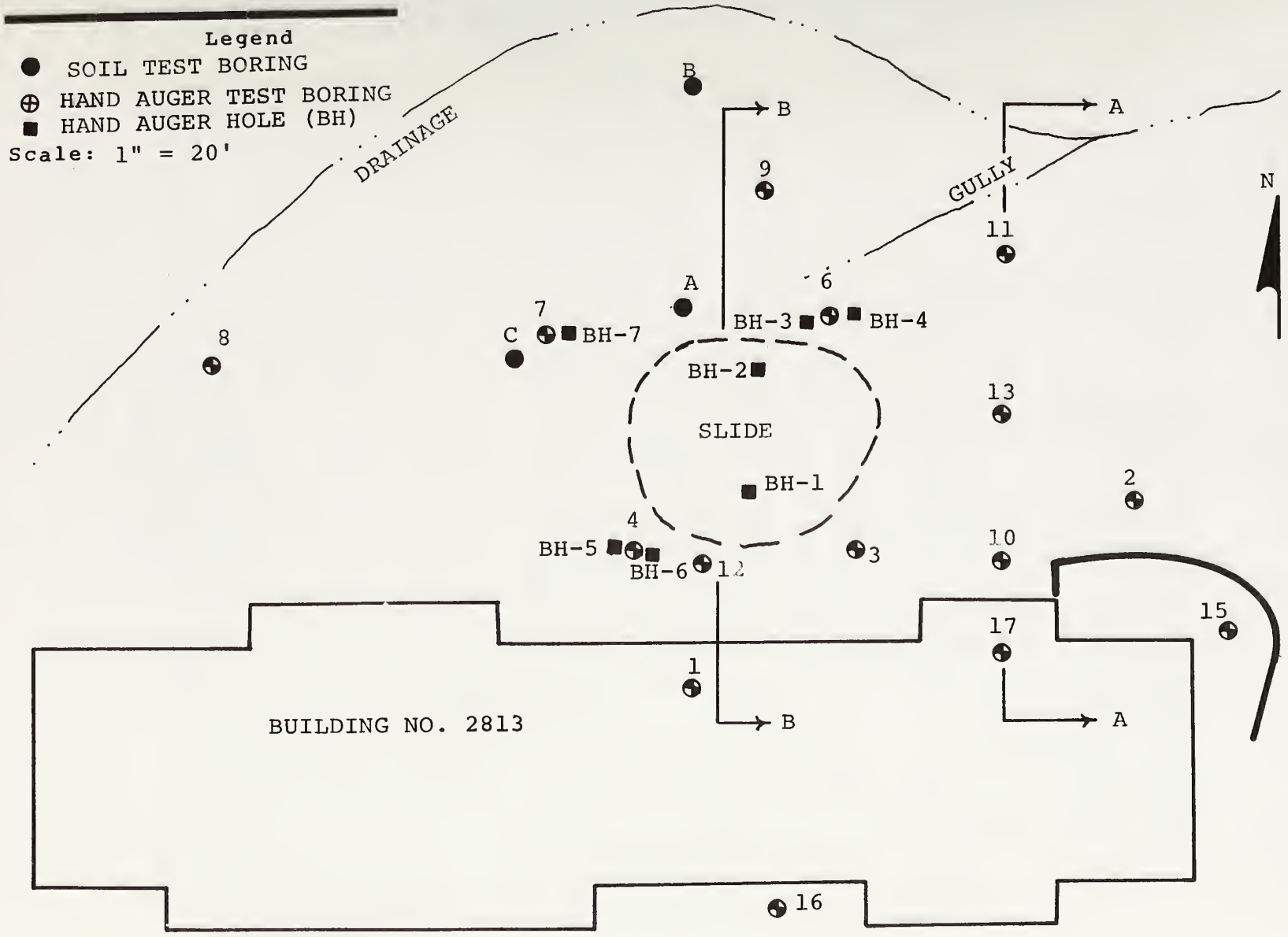


Figure 2 - Boring Plan, Chimney Hill Project

from approximately 2.1 to 4.0 meters (7 to 13 feet). Shallow ground water was not encountered in this area at the time the borings were made.

Shear Strength Evaluation: The shear strength of the soils in the vicinity of the landslide was evaluated by performing in-situ field shear tests with the Bore Hole Shear Apparatus. The results of the ten shear tests performed are given in figures 3 to 12.

Slope Stability Analysis: Slope stability analyses of the existing slope were performed for two generalized cross sections of the slope. Cross Section A-A was taken on the steeper 1.85(h) to 1.0(V) portion of the slope just to the west of the retaining wall. Cross Section B-B is located in the existing landslide area. The locations of these cross sections are shown on the Boring Plan, figure .

The generalized cross sections and soil properties used in the analysis are given in figures 13 and 14. The conditions used in the analysis were selected after carefully considering the very poor soil conditions encountered beneath the slopes, the great variability of these soils, and the possibility of loss of life should a larger landslide develop. These conditions are considered to be on the safe side, but certainly not unreasonable considering that one landslide had already occurred on the slope .

The simplified Bishop Method of stability analysis was used to estimate the safety of the slope. The slope stability analysis was performed with the assistance of a computer. The simplified Bishop Method has been found in the past to be generally superior to other commonly used methods. For purposes of the analyses failure of the slope was assumed to develop on circular surfaces with rotational movement occuring about the center of the circle.

The results of the stability analyses are summarized in Table 2. These results indicate that the side slopes behind the building could

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}	TURNS	
1	23.0	5.8	4.0	1.2	5	80
2	31.0	7.7	4.0	1.2	3	80
3	47.0	12.4	4.0	1.2	3	75
4	70.5	17.9	4.5	1.3	3	80

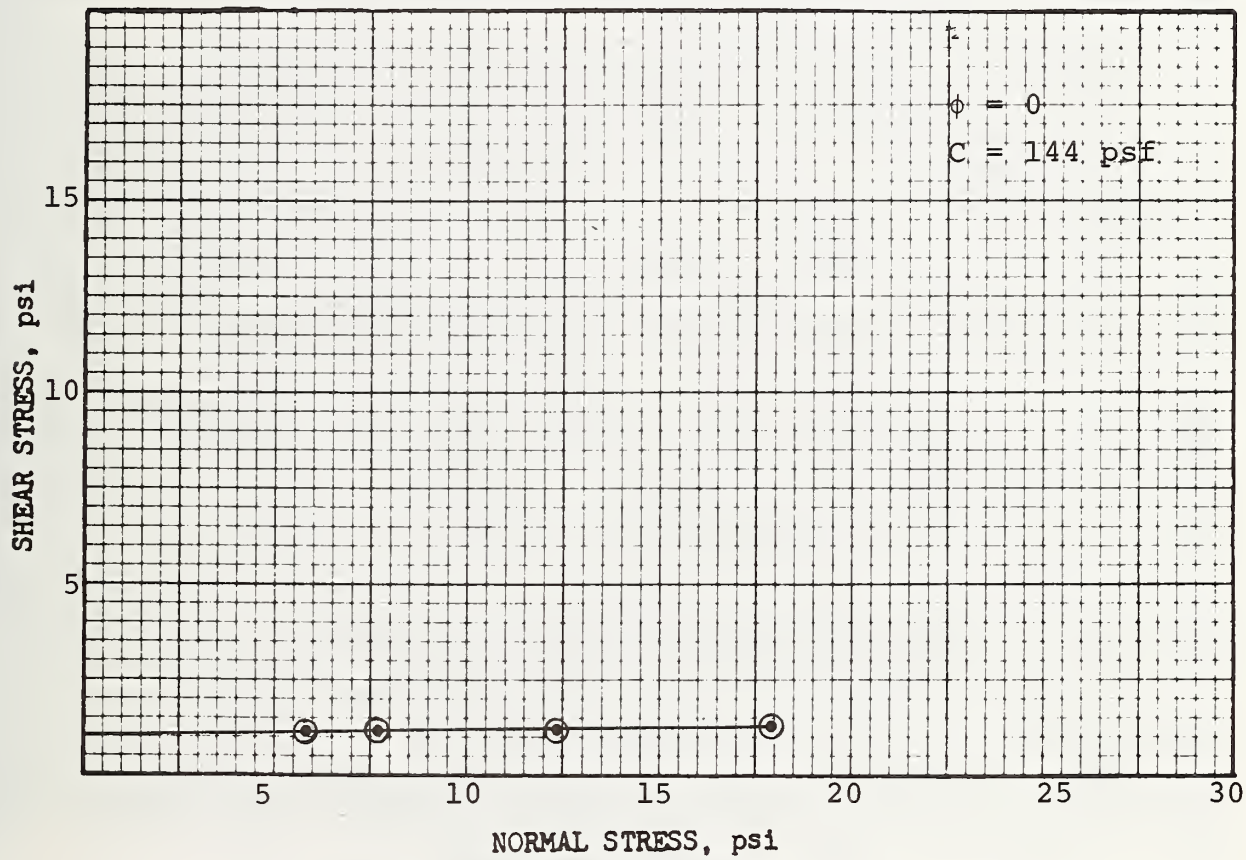


Figure 3 - BHS Shear Envelope, Bore Hole #2, 2 Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}		
1	21.0	5.0	14.3	3.9	5	153
2	30.5	7.7	21.0	5.5	3	130
3	40.0	10.1	26.5	7.0	3	135
4	50.0	12.7	33.0	8.4	3	131
5	60.0	15.2	39.0	10.6	3	140

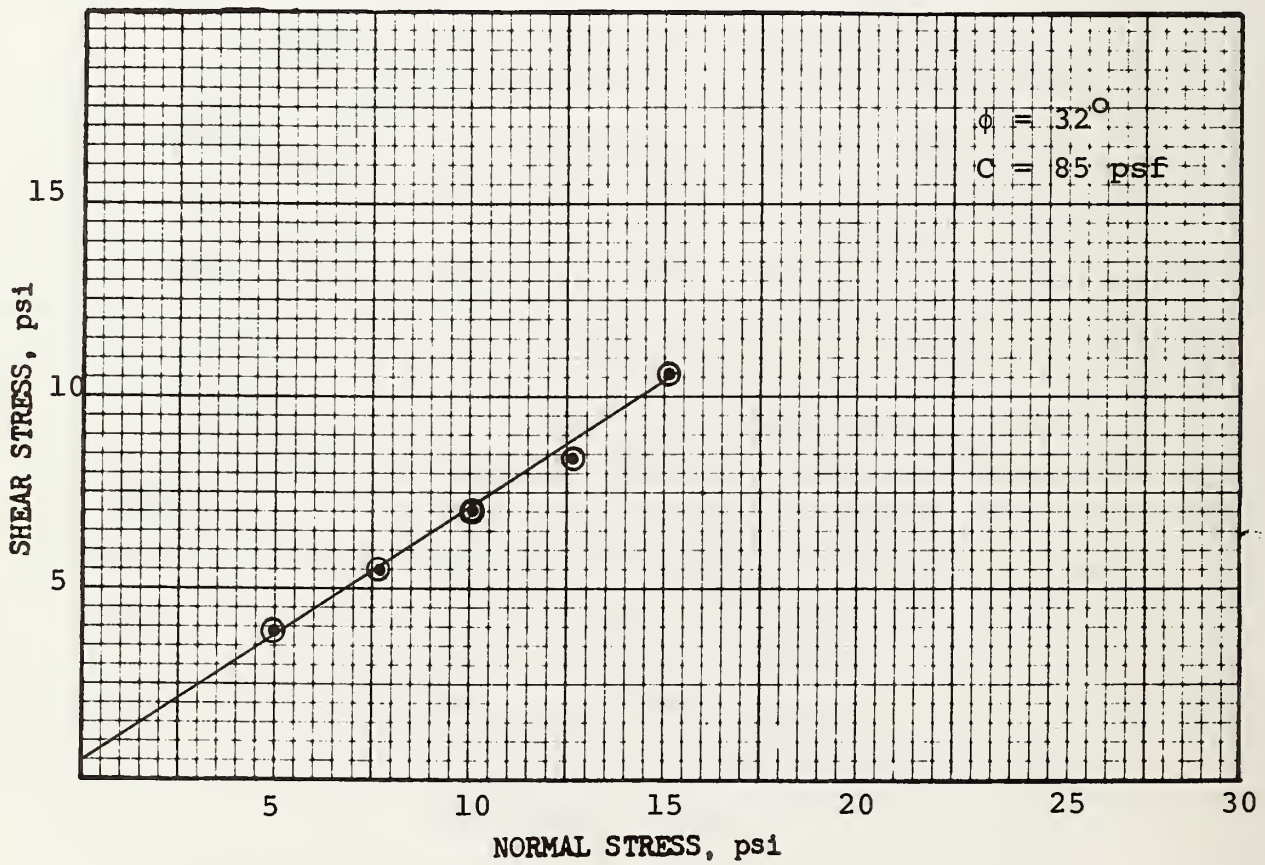


Figure 4 - BHS Shear Envelope, Bore Hole #1, 4½ Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}		
1	20	5	12	3.0	5	
2	40	10	17	5.0	5	
3	77	20	20	5.2	5	
4	117	30	21	5.3	5	

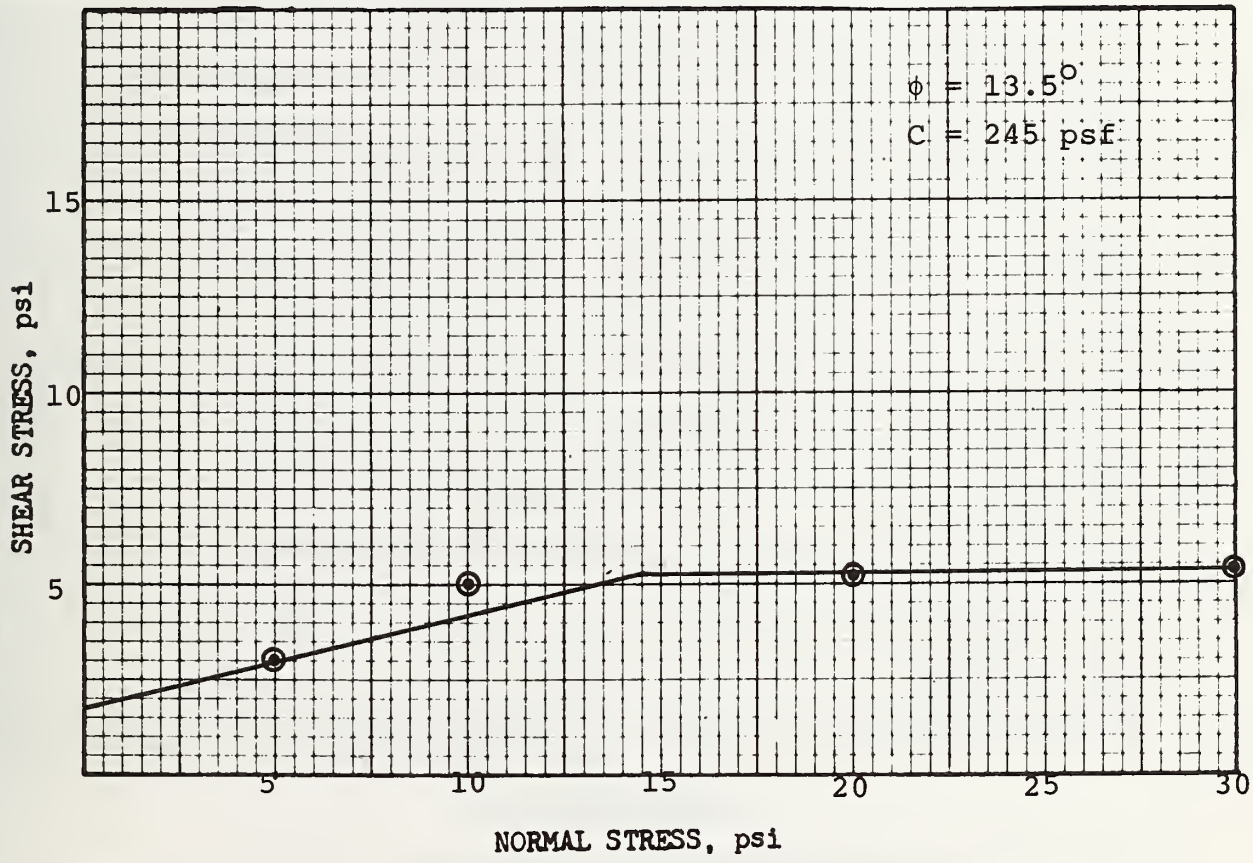


Figure 5 - BHS Shear Envelope, Bore Hole #3, 2 Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}		
1	20	5.0	7.0	2.0	5	
2	40	10.0	15.0	4.0	5	
3	59	15.0	18.0	5.0	5	
4	98	25.0	20.0	5.4	5	

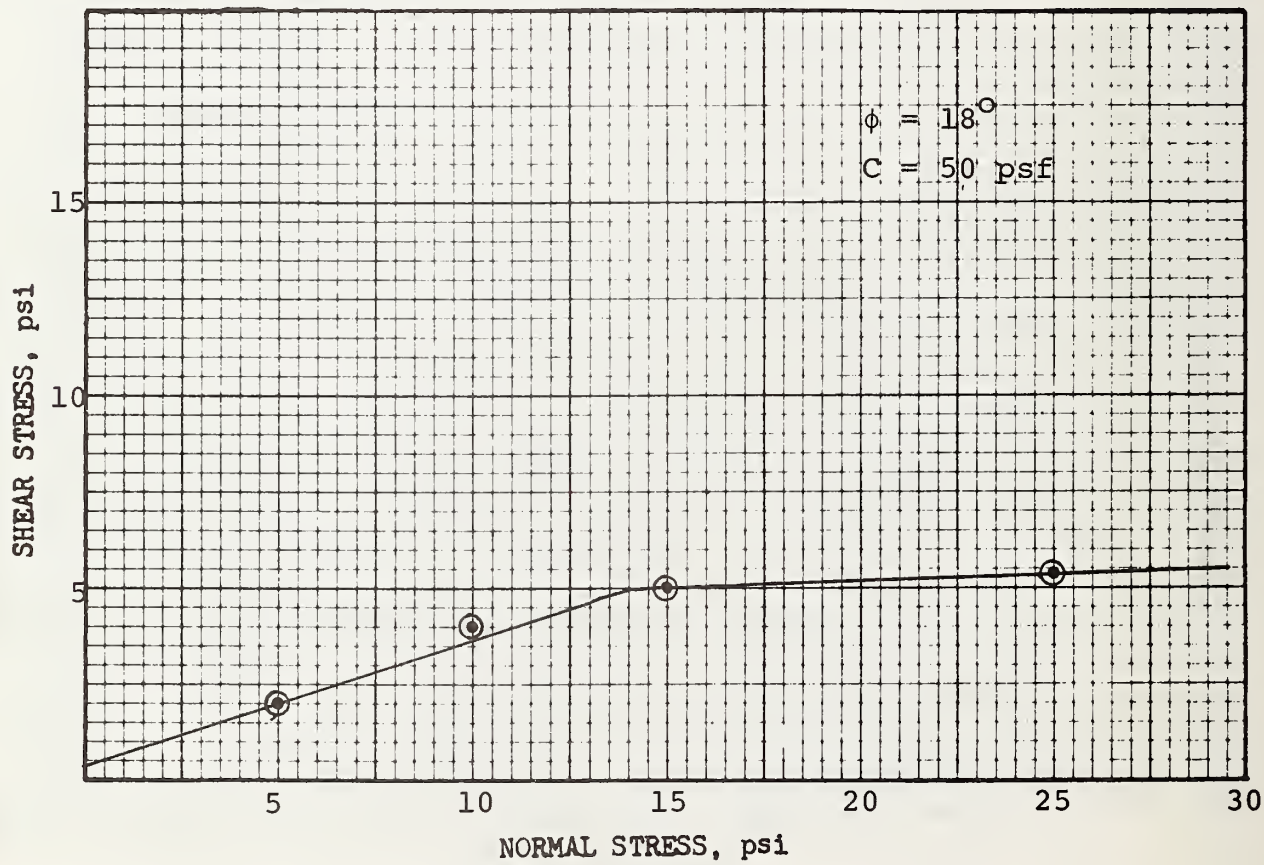


Figure 6 - BHS Shear Envelope, Bore Hole #3, 4 Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}		TURNS
1	20.5	5.0	9.5	2.6	10	120
2	31.5	7.6	19.0	5.2	10	214
3	44.2	11.2	25.0	6.7	10	159
4	56.0	14.2	27.0	7.3	10	165

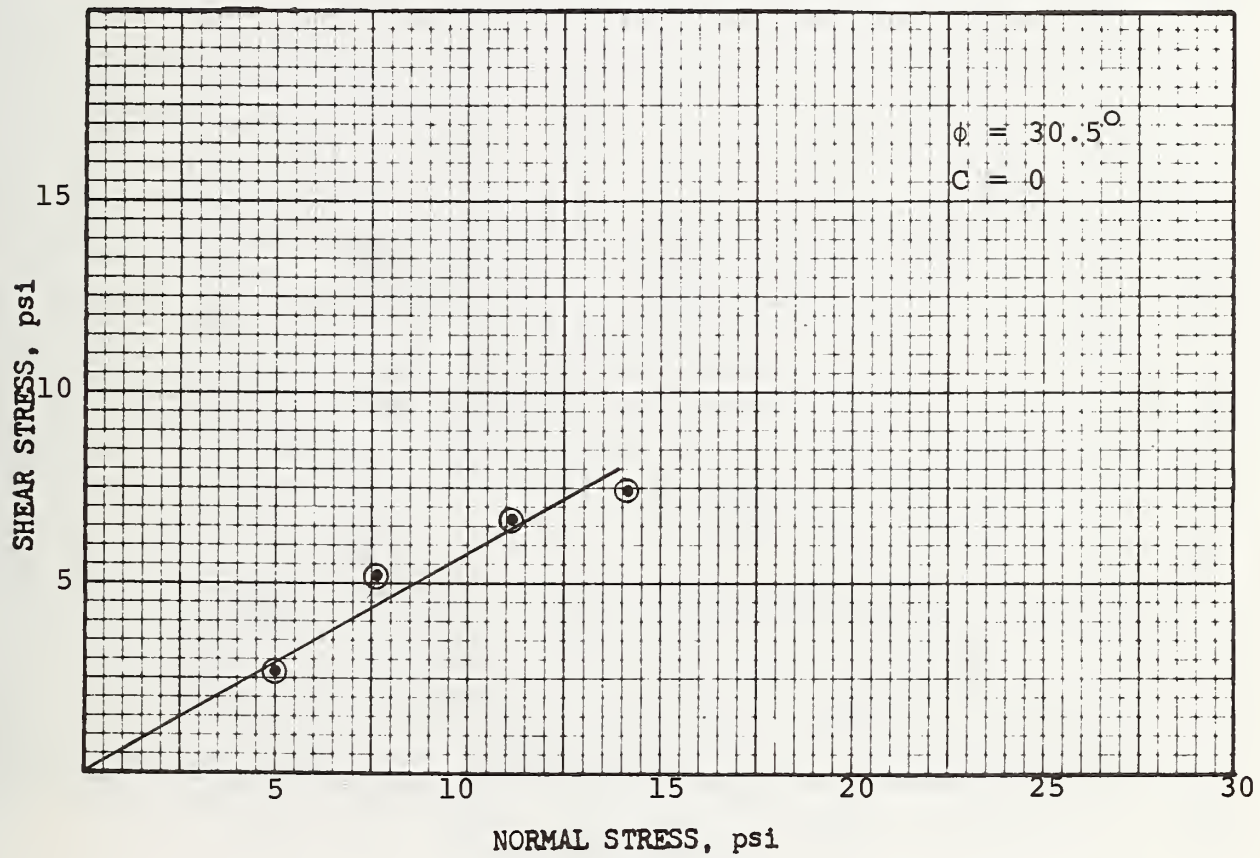


Figure 7 - BHS Shear Envelope, Bore Hole #4, 3½ Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}	TURNS	
1	18.5	4.6	12.0	3.1	5	180
2	30.6	7.8	18.0	4.9	5	80
3	41.0	10.2	24.0	6.6	5	85
4	51.2	13.0	30.5	8.1	5	115
5	61.8	15.4	35.0	9.3	5	120

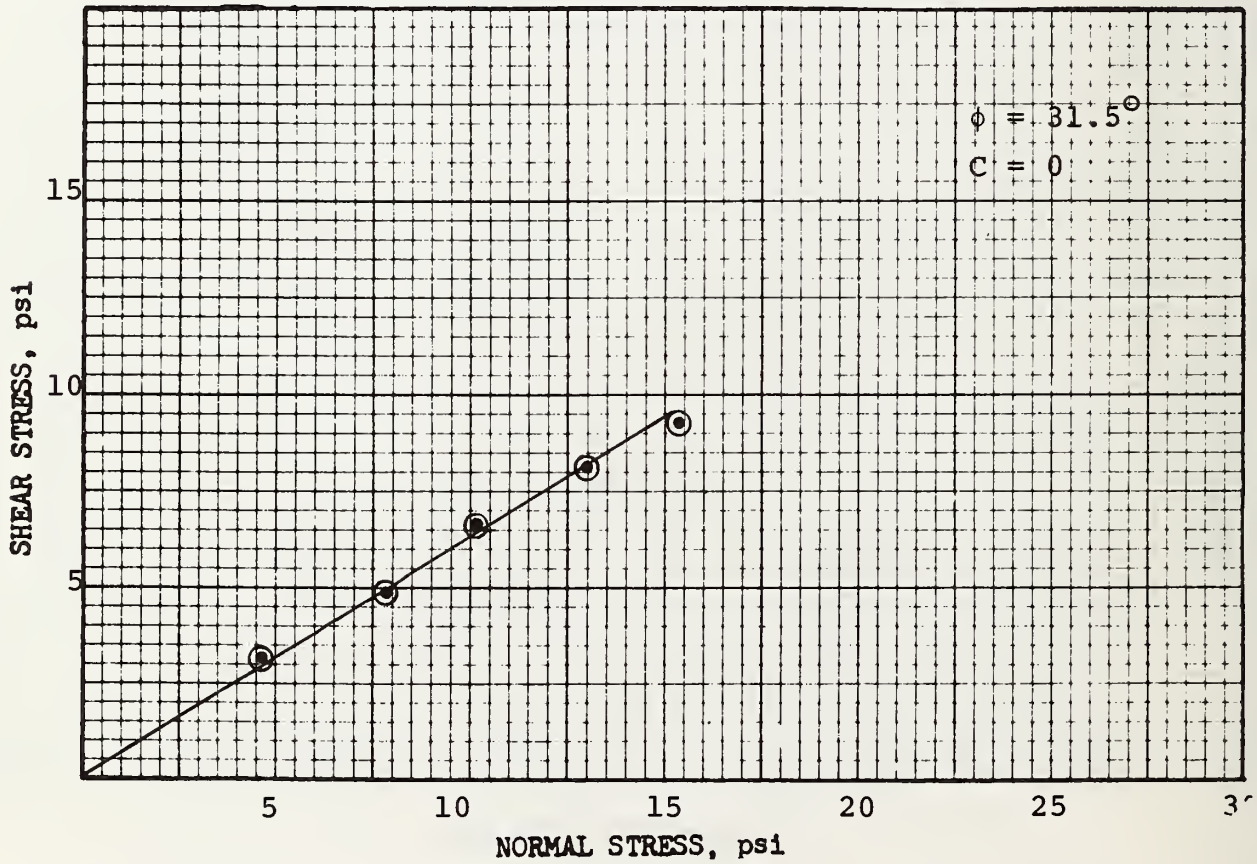


Figure 8 - BHS Shear Envelope, Bore Hole #4,
4 Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}		#TURNS
1	20.5	5.0	18.5	5.1	5	150
2	30.8	7.5	2.5	6.8	3	80
3	40.5	10.1	37.5	10.2	3	110
4	51.0	12.9	45.5	12.1	3	115
5	61.5	15.3	53.0	13.9	3	121

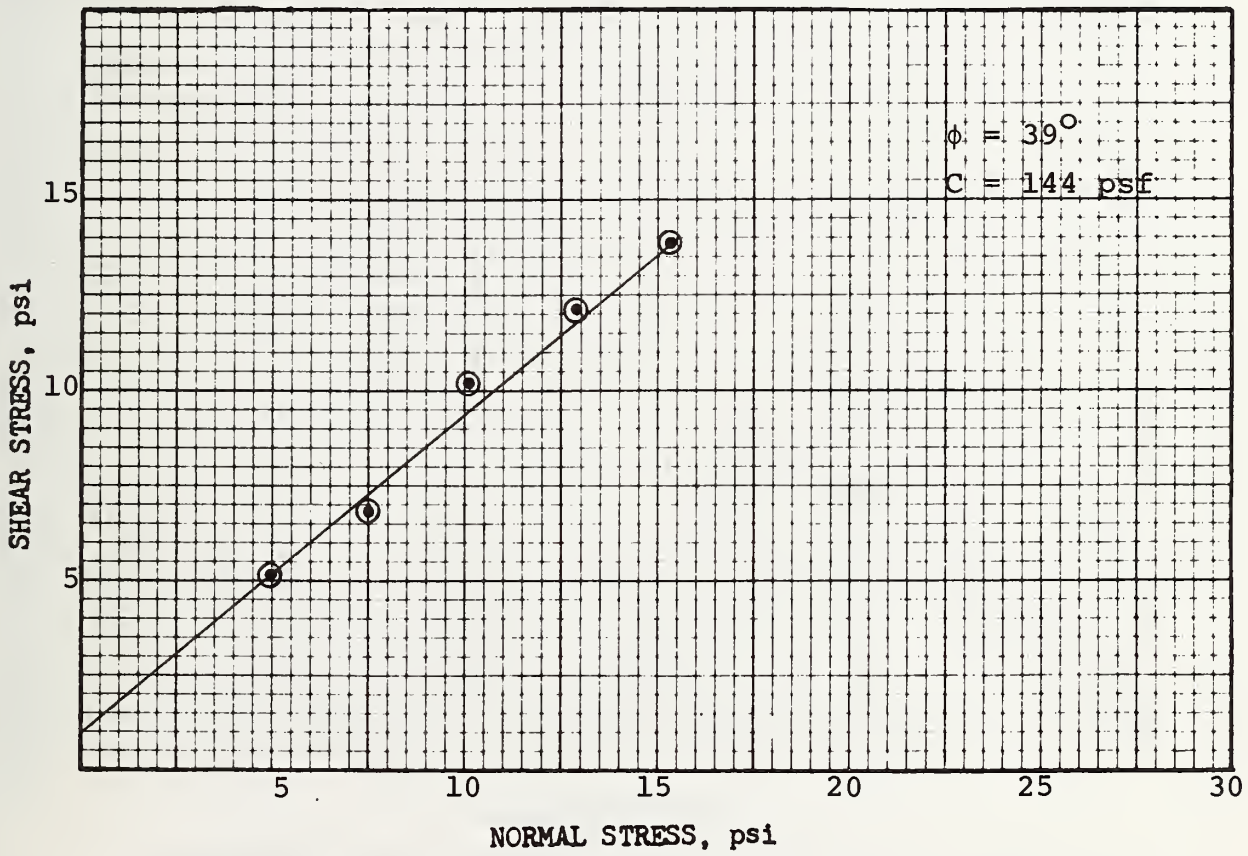


Figure 9 - BHS Shear Envelope, Bore Hole #5, 2¼ Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks TURNS
	Gauge	σ_n	Gauge	τ_{max}		
1	21.3	5.0	15.0	4.1	5	115
2	31.0	7.6	23.0	6.4	3	108
3	40.0	10.1	30.5	8.1	3	114
4	51.3	12.9	37.5	10.1	3	120
5	60.5	16.4	42.5	11.0	3	120

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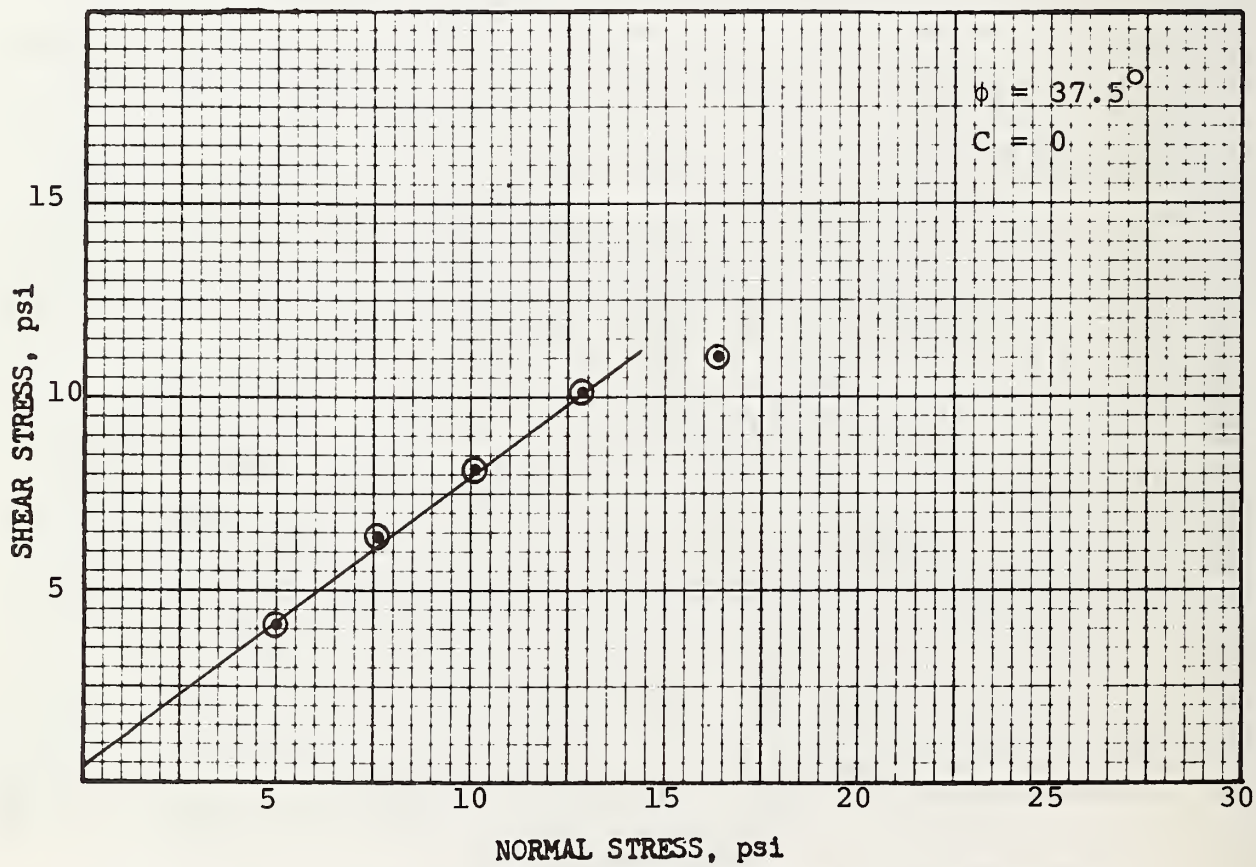


Figure 10 - BHS Shear Envelope, Bore Hole #6, 2¼ Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	Remarks
	Gauge	σ_n	Gauge	τ_{max}		
1	20	5		2.5	5 min.	
2	40	10		5	5 min.	
3	77	20		5.4	5 min.	
4	117	30		10.8	5 min.	

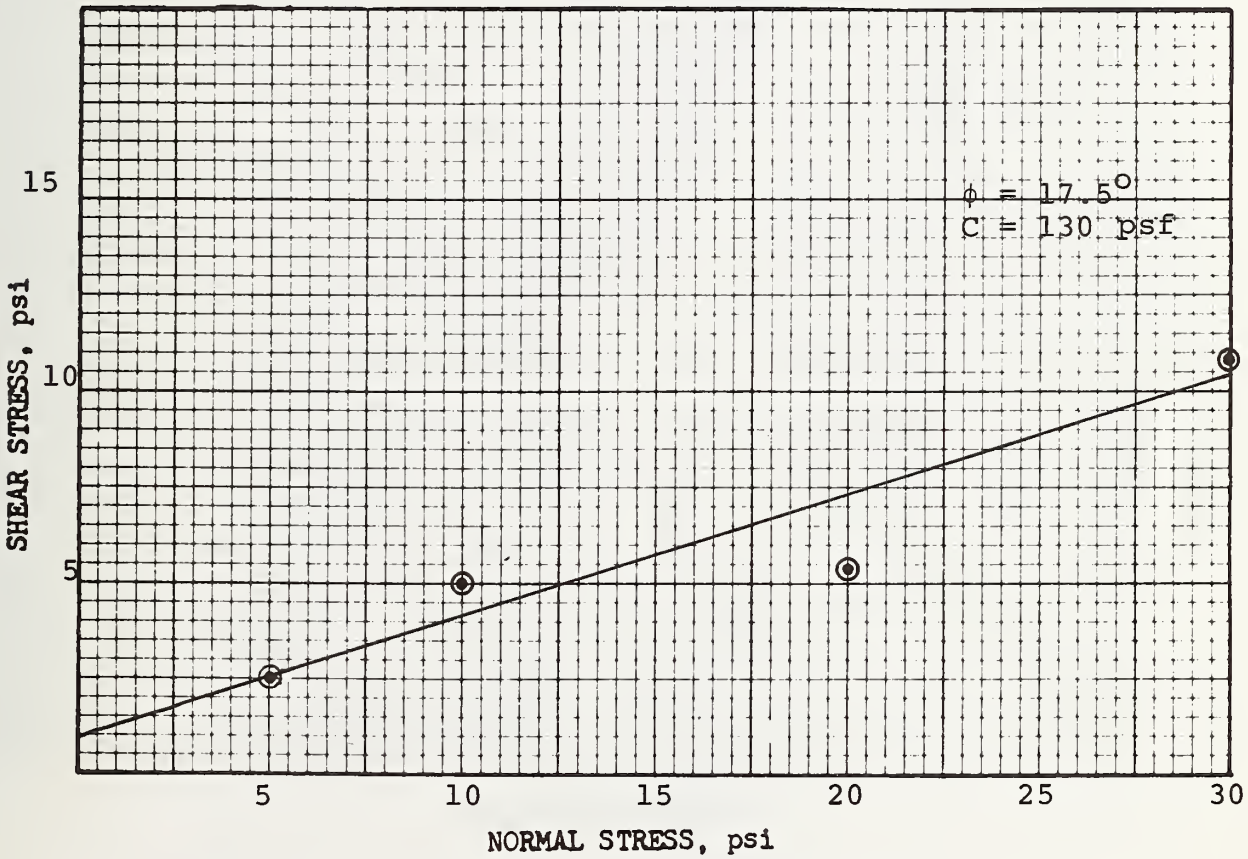


Figure 11 - BHS Shear Envelope, Bore Hole #7, 2 Feet Deep

Point No.	Normal Stress		Shear Stress		Cons. Time	TURNS	Remarks
	Gauge	σ_n	Gauge	τ_{max}			
1	20	5.0	17.0	4.5	5	102	
2	30	7.5	25.0	6.9	5	77	
3	40	10.0	34.0	9.2	5	91	
4	49	12.5	42.0	11.0	5	114	
5	58	15.0	51.0	13.4	5	119	
6	68	17.5	62.0	16.2	5	130	
7	78	20.0	70.0	18.5	5	131	
8	88	22.5	79.0	21.0	5	139	
9	98	25.0	87.0	23.3	5	136	
10	117	30.0	102.0	27.0	5	159	

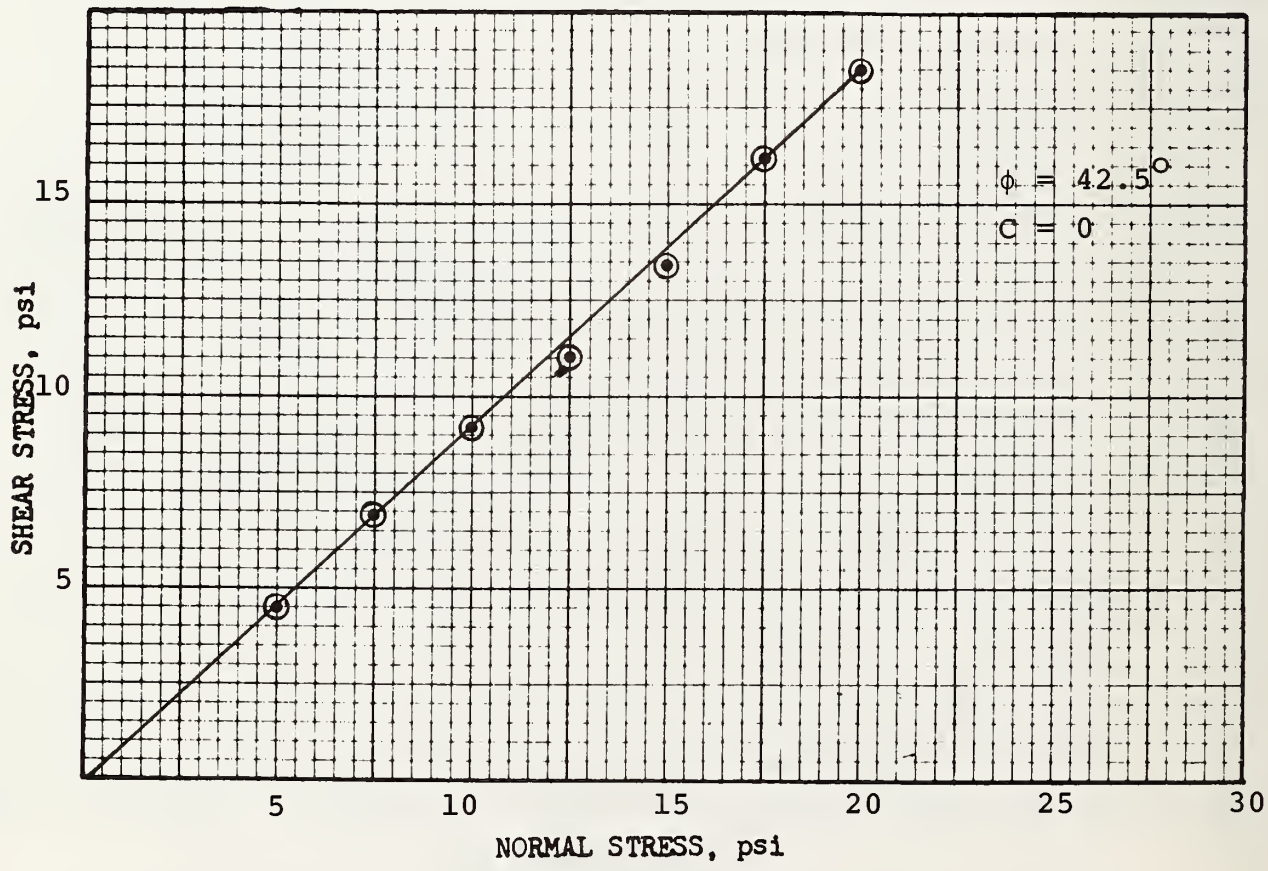
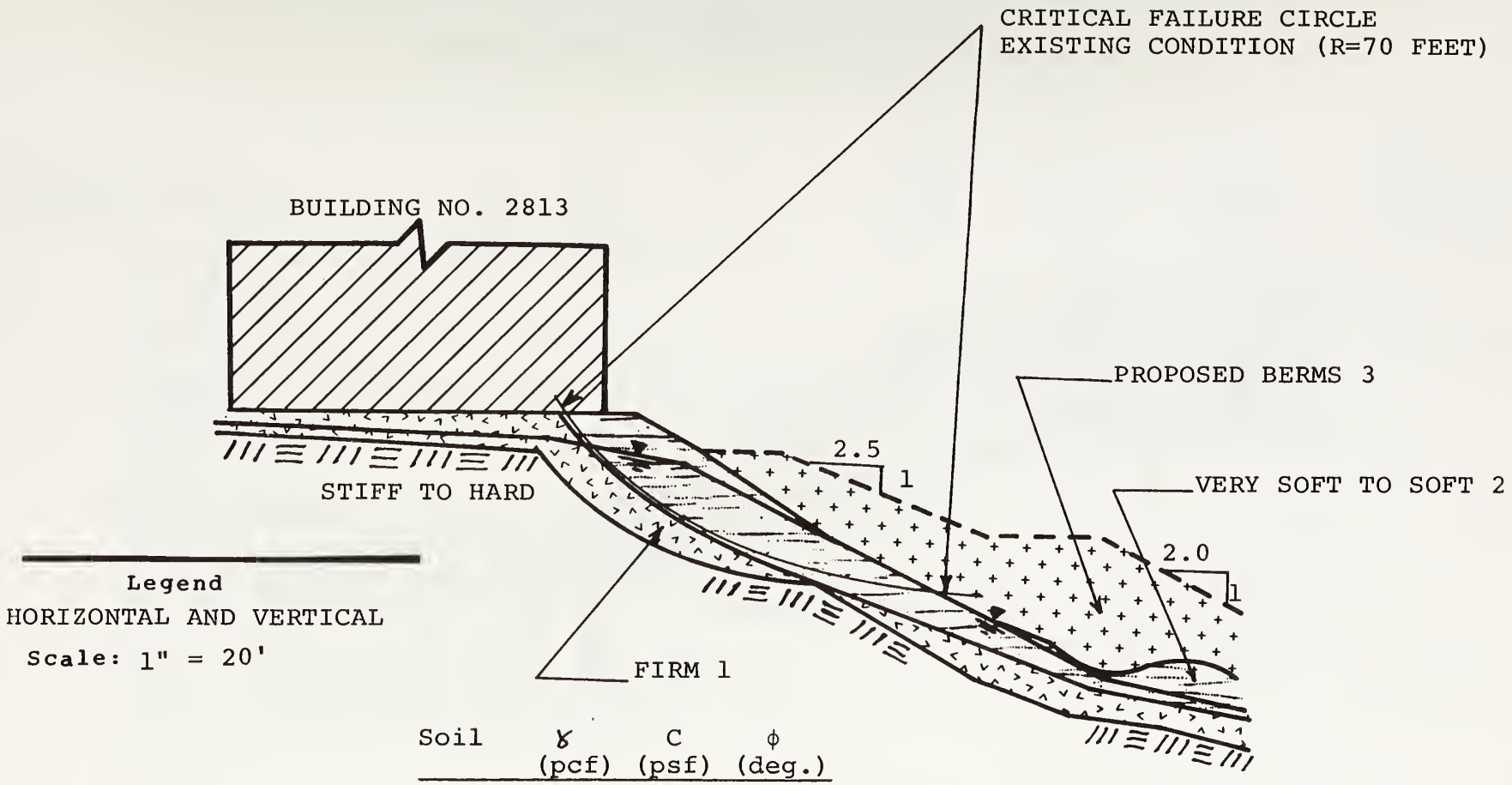
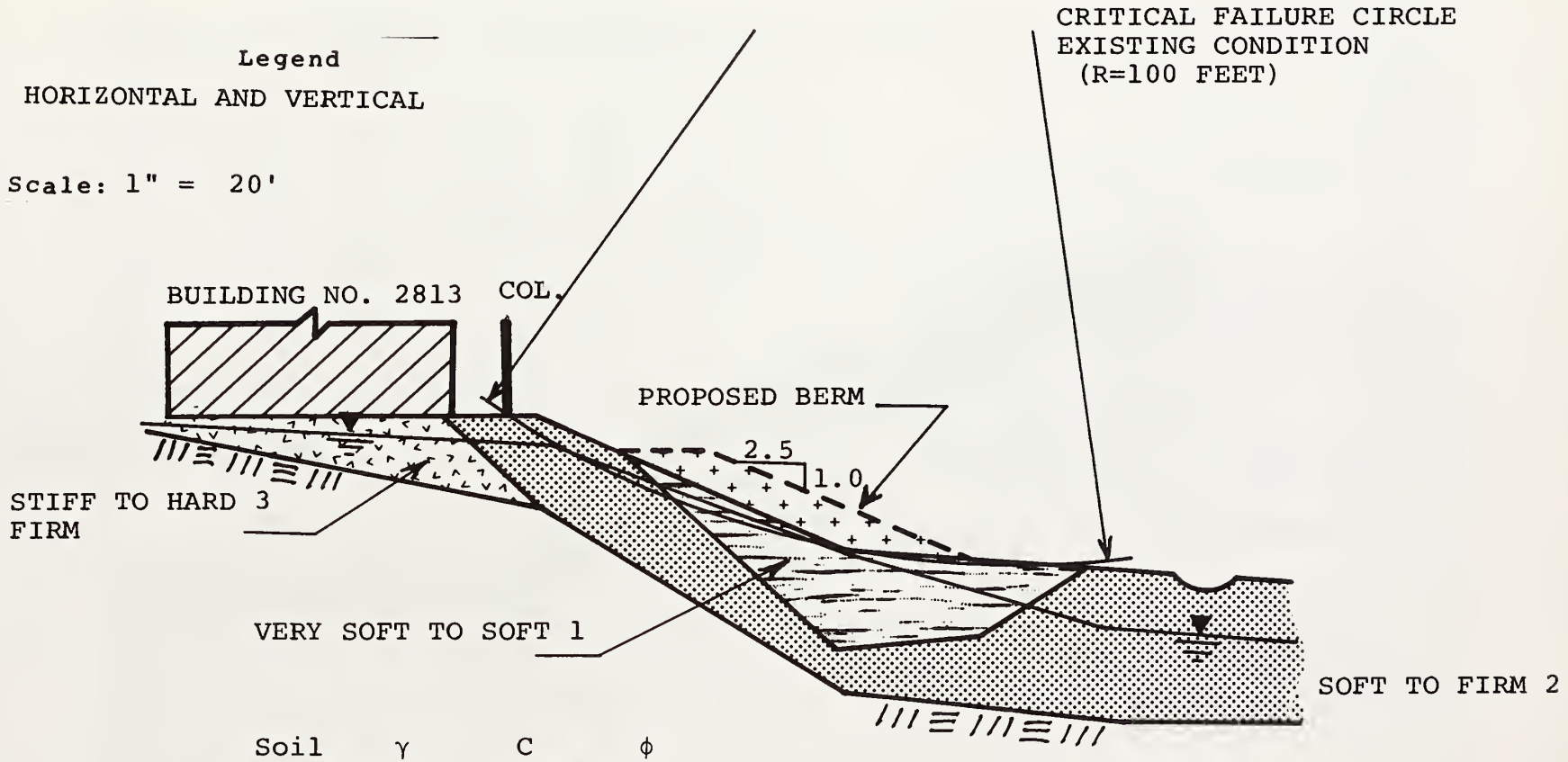


Figure 12 - BHS Shear Envelope, Bore Hole #7, 4 Feet Deep



Soil	γ (pcf)	C (psf)	ϕ (deg.)
1	124	216	0
2	124	288	26
3	124	288	26

FIGURE 13 - Generalized Section A-A, Slope Stability Analysis
Chimney Hills Project



Soil	γ (pcf)	C (psf)	ϕ (Deg.)
1	120	216	-
2	124	260	-
3	124	288	28

GENERALIZED SECTION B-B
USED IN STABILITY
ANALYSIS

Figure 14 - Generalized Section B-B, Slope Stability Analysis
Chimney Hill Project

TABLE 2

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Section	Assumed Slope Condition	Theoretical Location of Center of Failure Circle X Cord. (Feet)	Y Cord. (Feet)	Radius (Feet)	Safety Factor (Simplified Bishop Method)
A	1. Existing slope dry condition (better properties)	94	110	72	1.42
	2. Existing slope in a very wet condition	94	110	70	0.93
	3. Existing slope (wet) with 10 ft. wide berm 4 feet down from crest; 2.5(H):1.0(V) slope down vertically 10 feet and then a 2:1 slope	109	136	100	1.01
	4. Existing slope (wet) with 10 feet wide berm 4 feet vertically down from the crest at slope 2.5(H):1.0(V); second berm 20 feet wide and 14 feet vertically down from crest of existing slope with 2.0(H):1.0(V) slope	115	128	96	1.70
	5. Existing slope (wet) with 10 feet wide berm 4 feet vertically down from the crest at a slope of 2.5(H):1.0(V); second berm 15 feet wide and 14 feet vertically down from the crest of existing slope with 2.0(H):1.0(V) slope	115	128	96	1.50
B	1. Existing slope in a wet condition	97	128	100	0.84
	2. Existing slope (wet) with a single 10 feet wide berm 4 feet vertically down from the west at a slope of 2.5(H):1.0(V)	104	116	100	1.40

once again become unstable and slope stability failure could result. The theoretical critical failure surface for Section A-A for the assumed worst conditions would begin just inside the wall footings of the apartment building and extend down to the bottom of the soft layer. The maximum theoretical depth of the failure circle would be approximately 2.4 meters (8 feet). For this condition the safety factor during a wet period would be approximately 0.93.

The theoretical critical failure surface for Section B-B would also be relatively shallow. The surface would begin near the inside of the balcony footings and extend downward below the surface of the slope at the deepest depth only about 1.2 meters (4 feet). The theoretical safety factor for this condition was determined to be 0.84. The occurrence of a relatively shallow landslide appears to be in general agreement with the observed shallow slide which developed on the slope. The relatively large load from the balcony piers together with the poor subsurface conditions contribute to the low safety factor at Section B-B.

Discussion: The slope failure behind Building Number 2813 was probably caused primarily due to the soft, weak condition of the soil and by the presence of a significant quantity of water in the vicinity of the failure. The fill soils beneath the slope were either not compacted at all or were very poorly compacted. The wet condition of a significant portion of the fill in the area of the failure appears to have been directly related to the unusually heavy rainfall which occured in the Atlanta area during the month of August, 1974. During the 33 days preceding the failure 17.1 centimeters (6.75 inches) of rain fell at the Hartsfield International Airport; a similar amount of rainfall would be expected to have fallen in the vicinity of the Chimney Hills Apartments, approximately 18 miles north of the Airport.

The water in the slope appeared to have come from normal rainfall surface infiltration and also from subsurface ground sources. If surface rainfall in the immediate vicinity of the slope were the only source, water would not be expected to continue to flow from the toe of the landslide for several days after the rain had stopped as was observed

in the vicinity of the landslide.

At the time the hand auger borings were begun, a faucet located on the south side of the apartment building was observed to be dripping water. This faucet was just off-line from the slide area (on the opposite side of the building). Although the water from this faucet may have somewhat aggravated the problem, it was not felt that the leak was the main contributing factor causing the landslide.

Considering the results of the slope stability analyses and the fact that one slope failure has already occured, it was likely that other slope failures could develop in the future. A slope failure would be most likely to develop after a period of heavy rainfall such as the one which preceded the slope failure which just occurred.

The theoretical studies indicated that for the generalized worst subsurface conditions assumed to exist in the vicinity of sections A-A and B-B, the failure surfaces would extend to slightly within the building limits. However because of the non-homogeneity of the soil, the occurrence of localized soft and wet areas and because of other factors, the initial failure might not actually extend to within the limits of the apartment building. Should a future slope failure not extend to within the building limits, the building would more than likely be left with improper support. As a result, a successive slope failure could develop shortly after the first failure which would involve the building itself.

The lower shear strengths measured in the field were used in the stability analysis. It was felt that use of the lower shear strengths were justified because of: (1) generally poor subsurface conditions encountered beneath the slope; (2) the fact that the worst conditions may not have been encountered by the limited number of borings made; and (3) the significant fact that loss of life could be involved if a general slope failure did occur in the future.

Recommendations: A system of berms and/or retaining walls were recommended to be immediately constructed on the north side of Building Number 2813 to increase the safety factor of the slope to an acceptable

level. It was recommended that the specific remedial approach consider available materials and actual construction costs.

It was further recommended to increase the safety factor of the slope to an acceptable level by the use of soil berms. This alternative was recommended because of the apparent availability of soil fill. The poor foundation conditions in the vicinity of Section B-B would make the construction of the single berm desirable in this area. An economic analysis of the two alternatives was alternatives was recommended before a final decision is reached.

If alternative 2 were selected, involving a retaining wall, recommendations were made for additional hand auger borings to determine the depth beneath the surface that the foundation of the wall will have to extend to in order to reach suitable soil. This wall should be designed to carry the imposed earth pressure loading.

Case History No. 3: Bent Creek Apartments

In April 1976, a study was performed of a slope failure adjacent to the Bent Creek Apartment complex in Atlanta, Georgia.[9] The slope which exhibited significant movements (on the order of several feet down the slope) consisted of a poorly compacted or uncompacted fill placed on an unprepared original ground surface. The study was concerned with the causes of the then existing stability problems and the remedial measures necessary to prevent future slope failure.

The primary objective of the study was to determine the causes of slope movements on the east side of building No. 2040 and to evaluate and recommend feasible remedial measures. The investigation included the following specific objectives:

(1) To investigate the existing partially failed retaining wall.
(2) To determine the existing subsurface soil, water, and rock conditions in the slope area.
(3) To evaluate the shear strength of soils in the slope area.
(4) To anaylse the existing slope in order to determine the failure mode and probability of future movements of the slope.
(5) To identify and evaluate alternative remedial measures.
(6) To discuss alternative remedial measures and to recommend the

most practicable approach from a standpoint of effectiveness, economy, safety during construction and aesthetics.

(7) To make recommendations concerning construction procedures and quality control measures.

Bore Hole Shear data were used in slope stability analyses. Based on the slope stability analyses and conditions encountered, alternative remedial measures were analyzed. Eventually, the reinforced earth retaining wall structure was constructed.

Four years subsequent to the investigation, and about three and one half years since the wall was constructed, the slope has remained stable, and no further failures have occurred.

A relatively complete description of the study is presented below.

Field Investigation: Nineteen soil borings were performed at approximate locations as shown on the boring plan, figure 15. Eight of these soil borings consisted of hand augering and taking soil samples at approximately one foot intervals in order to determine soil types and the location of the interface between fill and residual soil. Two soil test borings consisted of machine drilling using the standard penetration test. These two soil test borings also consisted of machine augering to refusal. Refusal materials encountered during mechanical drilling may consist of boulders, thin rock lenses, or the upper surface of massive bedrock. The refusal materials encountered in these two soil test borrings represent the upper surface of massive rock. Nine of the soil borings consisted of different soil strata. These test borings were performed to hand auger refusal. Refusal (hand) is a term which denotes resistance of a magnitude sufficient to cause damage to the drilling equipment if additional progress were attempted. In the performance of hand augering, refusal material may consist of coarse gravel, cobbles, boulders, thin rock lenses, the upper surface of relatively massive rock, large tree roots, or man-made materials such as bricks, cinder blocks or segments of pipe. Due to the large amounts of brick and cinder block noticed on the surface of the fill, any refusal which occurred in the fill material was suspect of being possible debris. Bore Hole Shear testing was performed in one of the hand auger bore holes. Five BHS tests were performed in different strata within this boring.

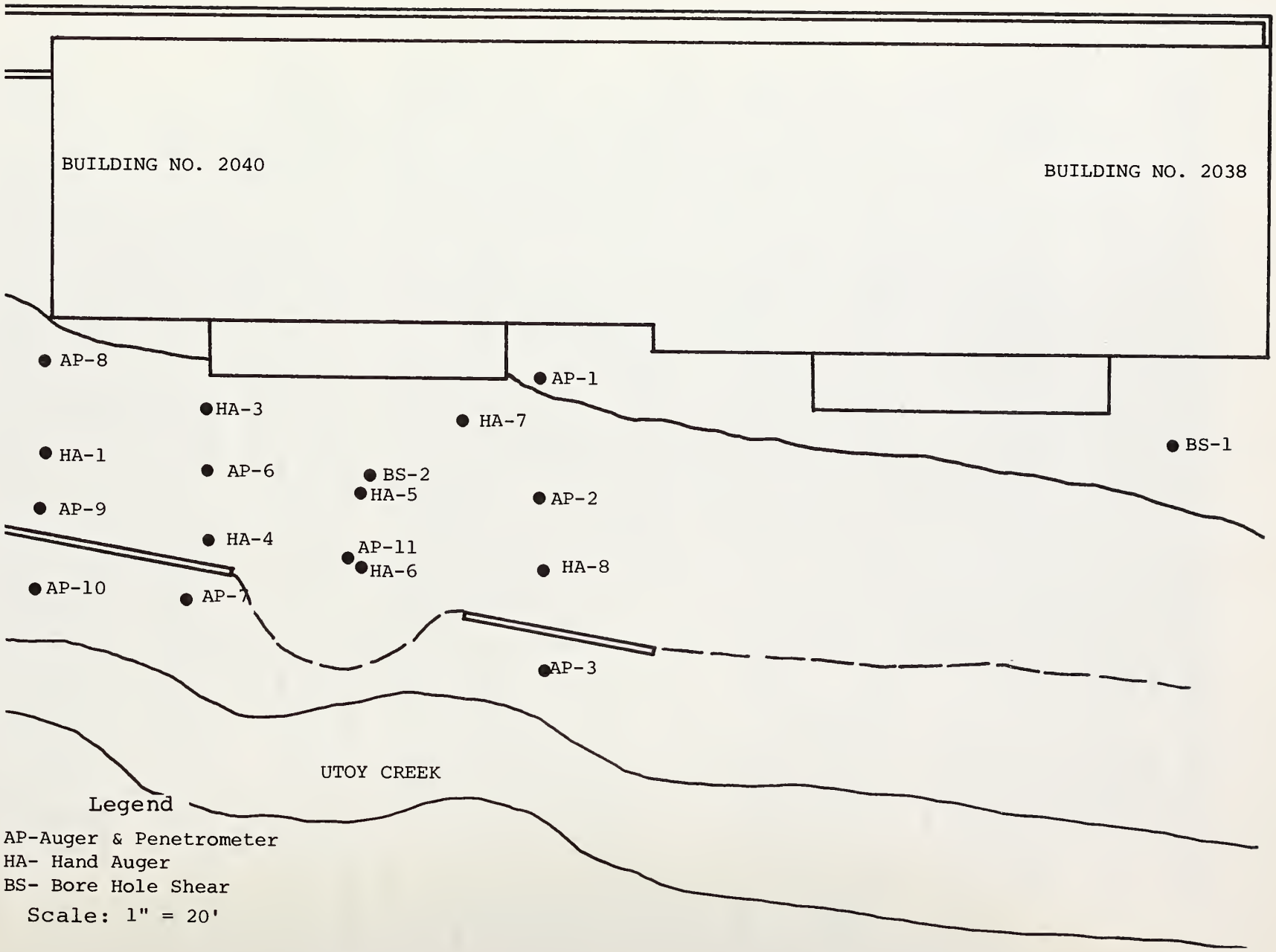


Figure 15 - Boring Plan, Bent Creek Apartment Project

Water level readings were made in each boring at the time of drilling and at approximately 24 hours after the completion of the boring.

Soil samples were visually inspected and classified by the project engineer. Select samples were chosen for moisture content tests and loss on ignition tests (% organics).

Laboratory Investigation: The laboratory investigation included moisture content tests which were performed on soil samples taken in the vicinity of the interface material between the fill and residual soils. The highly organic soils which were found in the interface region were subjected to the loss on ignition test which determines the quantity of volatile (organic) materials in the soil sample. These tests were performed as an aid in determining the general condition of the soil in the interface region. No laboratory shear strength tests were performed because of the difficulty of obtaining samples, and the confidence the author had in the reliability of shear strength data obtained.

Site Conditions: The site consisted of a four-story apartment building which was founded on caissons (drilled piers). Parking lots occupy the north and west sides of the building. The subject slope stability problem was situated on the east side of the building. At this location slope gradient varied from approximately 1.0 (H); 1.0(V) to 1.7(H) :1.0(V).

The slope was well vegetated with grass and six large trees, one of which showed tilting indicative of slope movement. However, boards had been placed behind some trees in order to attempt to retard soil movement. These boards were severly bowed due to earth pressure and had accumulated as much as 1 meter (3 feet) of soil behind them.

At the top of the slope, soil movement had exposed three caissons: one at the southeast corner of the building and two at the southeast concrete porch slab. At the southeast corner of the porch slab, the soil had subsided approximately 1 meter (3 feet). Acoompanying this vertical soil displacement which has produced a separation of 5 centimeters (2 inches) between the caissons and the soil.

A 1.8 meter (6 foot) high, 36.6 meter (120 foot) long granite masonry wall existed near the toe of the slope. The maximum tilt of this wall was

48 centimeters (1 foot 7 inches) from the vertical. Failure of the wall had occurred along a 10 meter (33 foot) section, and had allowed considerable soil movement.

At the toe of the slope was Utoy Creek which posed potential construction problems by limiting the working area. This creek could result in future erosion at the toe of the slope and further aggravate the problem.

Subsurface Conditions: The subsurface soil profiles obtained during the soil test boring operation showed a zone of from 2 to 3.7 meters (7 to 12 feet) of poorly compacted or uncompacted fill soils to exist on the slope. This zone extended from the building to the existing retaining wall. Underlying the fill soil was a relatively low consistency virgin residual soil. Soil test borings indicated that the slope of the residual soil surface was approximately the same as the existing fill surface. At the interface between the fill soil and the virgin residual soil was a zone of highly organic soil. This organic soil was found to contain as much as 11% organic material by weight. This material would with time, decompose and leach out of the soil leaving a very low consistency stratum between the fill and the virgin residual soil. On the average, refusal was encountered at depths of 3.4 meters (11 feet).

The following evaluations and recommendations were based on the data obtained in the soil test borings, field investigations, Bore Hole Shear tests, laboratory tests, slope stability analyses, and experiences with soils and subsurface conditions somewhat similar to those found at this site.

Slope Stability: Results of slope stability analyses revealed that the existing slope was in a state of impending failure with respect to surficial sliding and deeper failures in the fill material. Factors of safety for the deep failure in the fill soil ranged from 0.85 to 1.01 depending on the actual BHS soil strength parameters used. Slope failures would be expected to occur after periods of heavy rainfall, although failure could develop at any time. Slope stability analyses employed the modified Bishop method. The BHS factors of safety compared well with the field observed surficial failure condition, which would indicate an actual factor of safety of 1.0 or slightly less. Shear strength parameters were subsequently utilized in design of stabilization alternatives.

Reinforced Concrete Retaining Wall: A reinforced concrete gravity retaining wall was evaluated for construction in the location of the existing retaining wall. Excessive costs limit the feasibility of this alternative. Other considerations unfavorable to this alternative were the excessive construction time required, stability problems during construction, and the safety to construction workers. The problem of construction time was especially critical in that the probability of rain during the construction period increased exponentially with the expected length of the construction period. The occurrence of rain during construction greatly increased the probability of a failure during the construction phase. The approximate cost estimate for a reinforced concrete retaining wall, 36.3 meters (120 feet) long at the time of the study, was $134,000.

Caisson Wall: A caisson wall was evaluated for construction in a location immediately east of the building. Considerations unfavorable to this alternative included possible interference with the building foundation and aesthetic deficiencies (lack of backyard). Safety was another negative aspect of this solution since the unstable soil could be expected to continue moving, and a steep escarpment was anticipated to form adjacent to the caisson wall from future soil movements. A rough cost estimate for the caisson wall at the time of the study was $55,000. It was concluded that this was not the best engineering solution to the problem.

Granite Masonry Gravity Retaining Wall: A granite masonry gravity retaining wall was evaluated for construction at the location of the existing retaining wall. The problems with this alternative were somewhat similar to those of the concrete retaining wall; long construction period, stability problems during construction and safety of construction workers. However, this wall unlike the reinforced concrete wall, could be constructed in sections, thus exposing only limited portions of a cutback slope to potential movements. The length of time to construct the segments, and hence the length of time the cutback slope portion was exposed, could be excessive. The total costs of this alternative were estimated at the time of the study to be on the order of $70,000 for a 36.6 meter (120 foot) wall length.

Reinforced Earth Gravity Retaining Wall: Reinforced earth is a patented technique in which a composite material is formed by the association of a

frictional non-cohesive soil with metal reinforcement. A structure constructed with reinforced earth behaves as a coherent gravity mass which reduces stress concentrations in the foundation soils and within its own mass. A reinforced earth structure is also capable of withstanding significant differential settlement without loss of structural integrity.

A reinforced earth retaining wall is constructed by the simultaneous placing of cohesionless soil or stone and metal strips in lifts to the required wall height. The exposed face of the wall is protected by a prefabricated concrete facing which is connected to the metal strips.

This alternative had a number of advantages and relatively few disadvantages for this particular situation. The ability to perform construction in a segmented manner is a definite benefit in terms of minimizing failure probability during construction. By performing the construction in segments, it was planned to take advantage of a phenomenon known as arching to reduce the possibility of slope failure. The reinforced earth retaining wall could conceivably be constructed in a period of one week, which appeared significantly less than the other alternatives. Safety of construction workers was an important consideration with this alternative in that they should not have to spend extended periods in the excavated areas. Finally this alternative would be aesthetically pleasing. The cost of the reinforced earth retaining wall had been estimated at the time of this study to be on the order of $68,000 for a 36.6 meter (120 foot) length. The reinforced earth structure was recommended as the best solution to the problem. The recommended wall was a 4.9 meter (16 foot) wide, 6.1 meter (20 foot) high, 36.6 meter (120 foot) long structure to be founded on rock or very hard or very dense soils as shown in the plan and profiles shown on figures 16 and 17. Additional information was obtained on the construction of the reinforced earth retaining wall from the Reinforced Earth Company, which provided materials and some technical advice. The site was inspected by their local engineer.

This alternative appeared to be a practicable solution which would significantly increase the slope stability. It appeared to be relatively

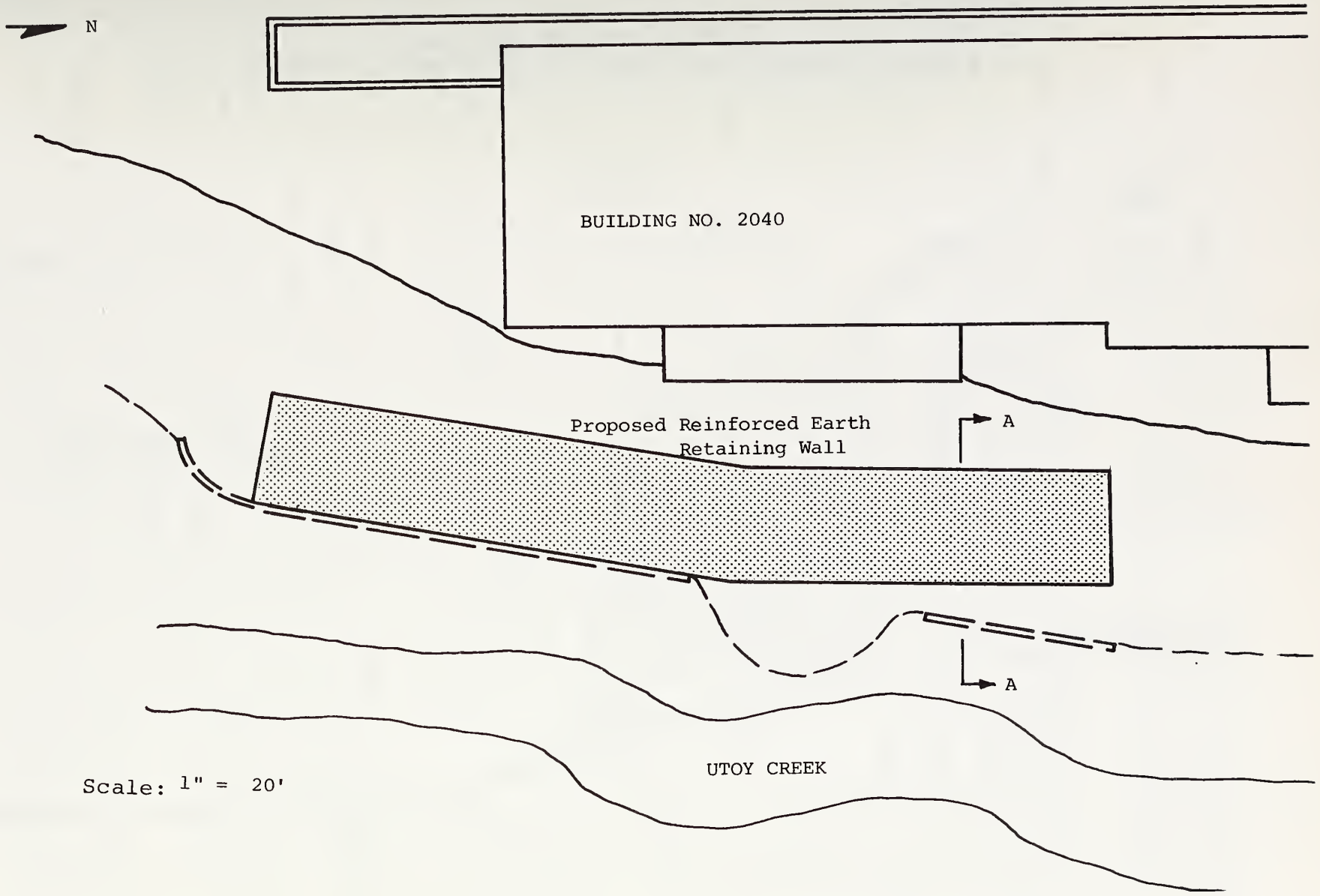


Figure 16 - Proposed Reinforced Earth Retaining Wall, Bent Creek Project

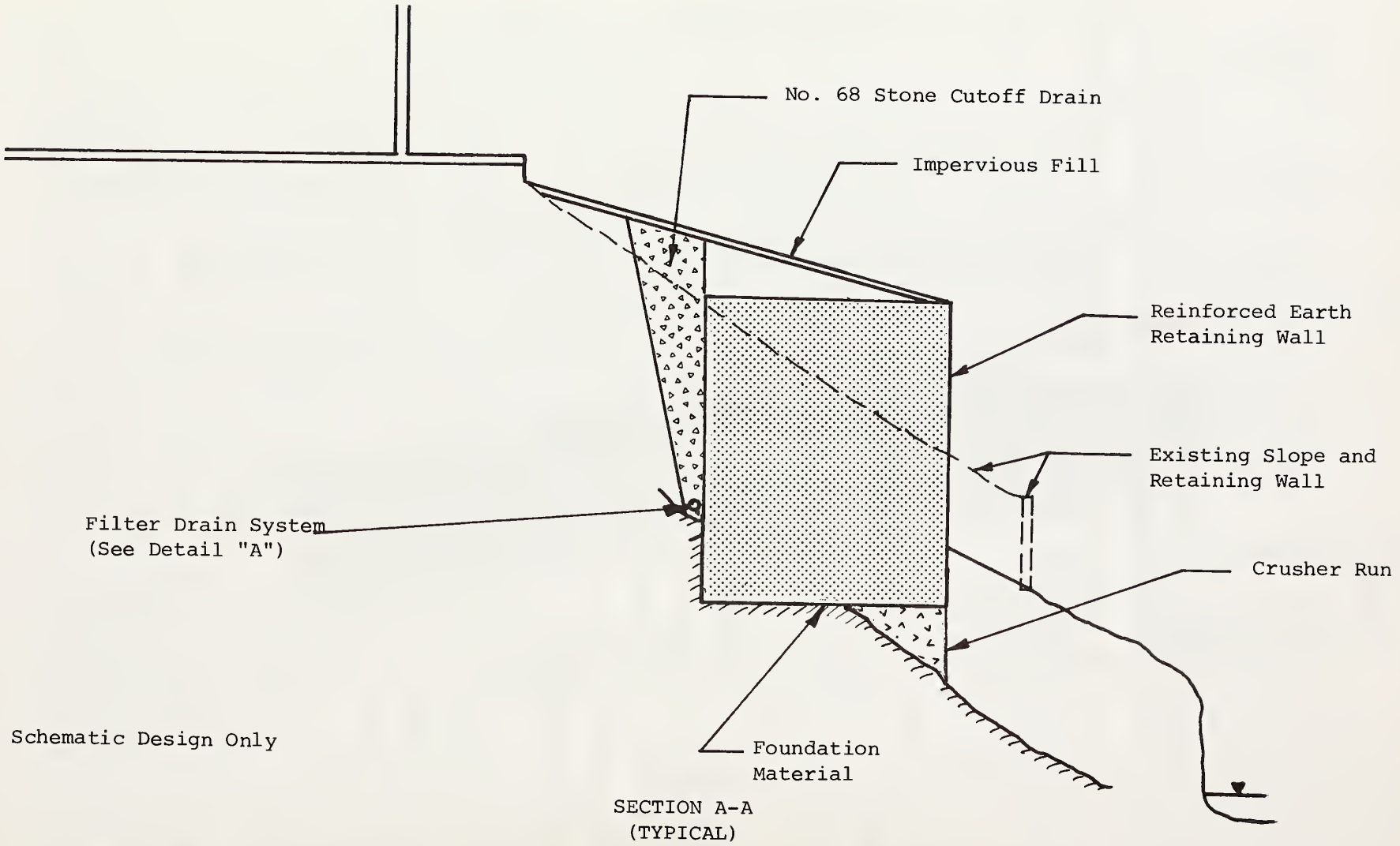


Figure 17 - Section A-A (Typical) Reinforced Earth Retaining Wall, Bent Creek Apartment Project

economical in comparison with other alternatives. It was therefore recommended for this project. The Reinforced Earth wall was constructed and the slope has performed adequately since construction was completed four years ago.

Case History No. 4 - Rich's Computer Center site, Georgia Tech Research

A special design study was performed in 1970 by Michael R. Eckmann[10] as a research design project in partial fulfillment to obtain a Master's Degree. This study was an evaluation of the Bore Hole Shear test in Piedmont residual soils. More specifically, the practicability of Bore Hole Shear testing as a method of obtaining meaningful shear strength parameters in hard and dense soils was investigated. Piedmont soils are residual soils found within the Piedmont Geological province in Georgia and North and South Carolina. They generally consist of an upper clayey zone, normally a clayey silt (CL), with underlying sandy silts and silty sands. Consistency of these soils generally increases with depth. They are often micaceous, with varying amounts of mica (from non-micaceous to highly micaceous). The soils are underlain by parent granites, gneisses, and schists.

The case history to be discussed is the Rich's Computer Center Site in Atlanta, Georgia. It should be pointed out that the Bore Hole Shear device used in this study was an early hydraulic model that predated the present commercial, pneumatic instrument. The accuracy of results obtained from this model were not of the caliber which are obtained in the improved device.

Test procedures at the Rich's Site were to conduct Bore Hole Shear tests in the same holes from which undisturbed samples were obtained. Personal supervision by the investigators insured that test depths and soils tested were similar, and that test procedures both for the laboratory triaxial tests as well as the BHS tests were proper. This was particularly critical in Piedmont soils, since significant changes in engineering properties often occur within narrow depth intervals.

The actual soils tested at this site were a general transition zone between dense micaceous silty sand and partially weathered rock. These granular soils were of relatively high consistency, and would classify as dense to very dense silty sands and/or hard to very hard sandy silts.

Sampling the soils for subsequent laboratory testing thus proved difficult and time-consuming. The site itself was a proposed site for the construction of a structure to house computer facilities and equipment. No structural information was available at the time of the study, but it was to be a masonry structure with brick exterior walls. The building was 2 stories in height, and no unusual column or wall loads were known. Information provided by laboratory and field investigations would be used in foundation evaluations and recommendations. The need for shear strength data for foundation design was apparently not of high priority. However, actual strength testing of Piedmont soils within the partially weathered rock zone had been quite limited in the past due to difficulty in obtaining samples. Driving of thin-wall tubes often resulted in disturbance of the soil sample or damage to the tube itself. Data often used to estimate strengths of this very dense or very hard material are standard penetration blow count data. Penetration resistances in this zone usually range from 80 blows per foot to well over 100 blows per foot.

Bore Hole Number 1: Bore Hole Shear tests were conducted at depths of 50.8 centimeters (20 inches) and 35.6 centimeters (14 inches). After completion of the 20 inch test and upon completion of the 14 inch test, the device overexpanded, preventing it from being hydraulically closed. The hole was damaged while forcing the device up. This prevented further testing in the hole. These tests are plotted as tests on numbers 1 and 2 on figure 18. Shear strength parameters of $C = 0$, $\phi = 31.0^\circ$ and $C = .280$, $\phi = 22.5^\circ$ were calculated from the BHS results. The triaxial test data are shown on figure 19. It was the judgement of the investigator at the time that the Mohr envelopes obtained from the two higher confining pressures were in virtually the same soil, while that obtained from the lower confining pressure (10 psi) represented a soil with different engineering characteristics. This was indicated by the close agreement in void ratio and moisture contents of the two samples at the higher confining pressures, and also more closely represented the depths of the BHS tests in this hole. A comparison of the BHS enevelopes and triaxial envelop is shown as figure 20.

Bore Hole Number 2: Three tests with the Bore Hole Shear device were accomplished at depths of 48.2, 0.5 and 15.2 centimeters (19, 12 and 6 inches) at the second field test location. BHS test data are

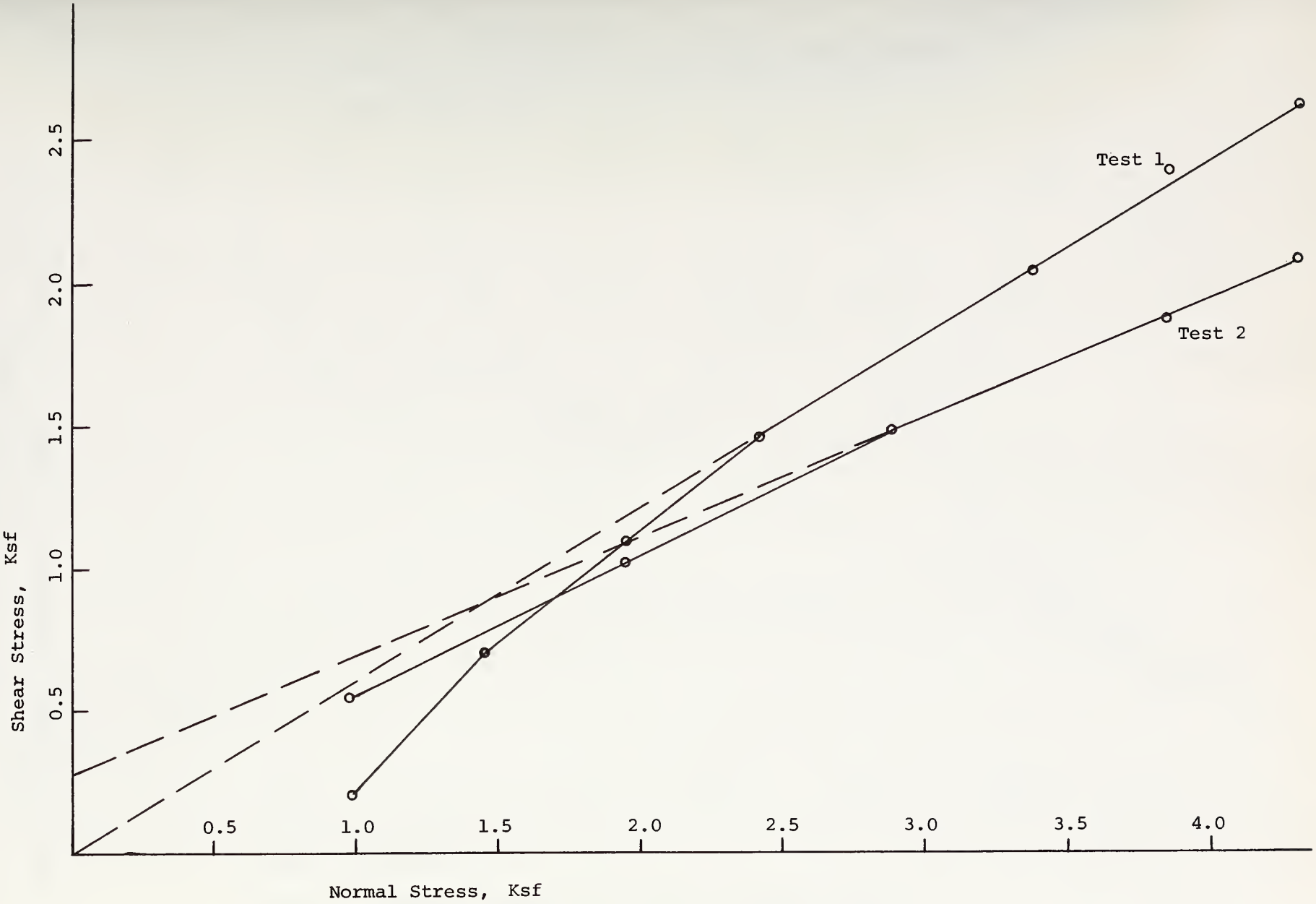


Figure 18 - BHS Data, Rich's Computer Center
Bore Hole #1

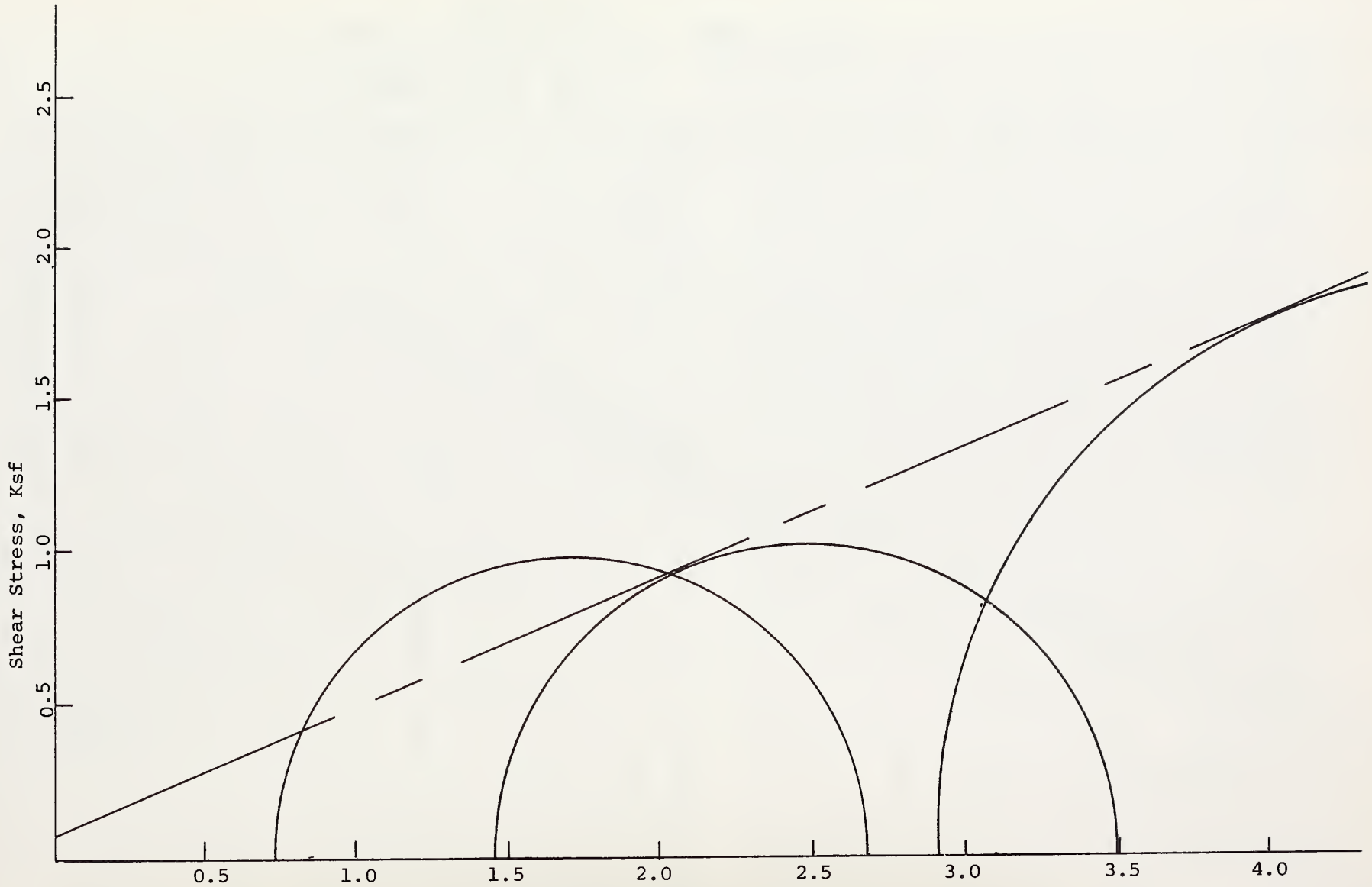


Figure 19 - Triaxial Shear Test, Rich's Computer Test
Bore Hole #1

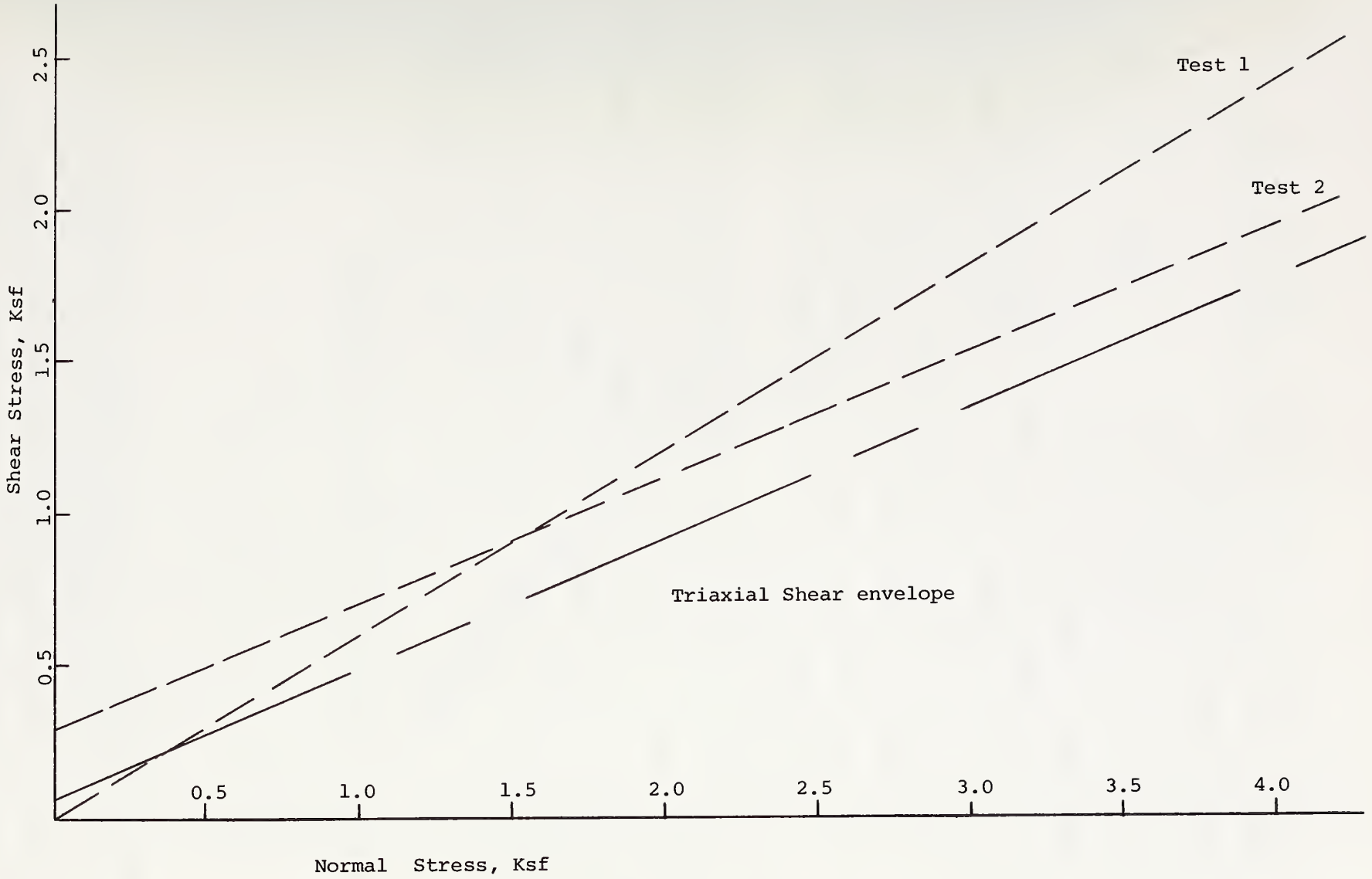


Figure 20 - BHS and Triaxial Data- Rich's Computer Center
Bore Hole #1

plotted as tests number 1, 2, and 3 on figure 21. The triaxial shear test (unconsolidated, undrained) is plotted on figure 22. Interpreted Shear Strength parameters are as follows:

BHS Test No. 1 - C = 480 psf; ϕ = 29.5°

BHS Test No. 2 - C = 330 psf; ϕ = 29.0°

BHS Test No. 3 - C = 75 psf; ϕ = 17.5°

Triaxial Test - C = 420 psf; ϕ = 17.5°

The triaxial shear envelope was plotted using the higher two confining pressures because the top sample, tested at the lowest confining pressure, had been damaged. As a result, it was only 4.8 inches long rather than the minimum length of 5.6 inches required by ASTM for 2.8 inch diameter sample. The triaxial test of this sample may have resulted in a higher shear strength than reasonable, because of the smaller length-to-diameter ratio. A comparative plot of the Bore Hole Shear device tests and triaxial test is shown on figure 23.

The "Undisturbed" sample for bore hole no. 2 was the most disturbed of the samples, and the soil was extremely difficult to prepare for triaxial testing. In addition to the one short specimen mentioned previously, another sample also contained a horizontal crack. Therfore, for the three boring locations tested, the data which are the most suspect are those from the second boring location.

Bore Hole Number 3: Tests with the BHS device were performed at depths 48.2, 30.5 and 15.2 centimeters (20, 12 and 6 inches) in this boring. Results are plotted as tests no. 1, 2 and 3 on figure 24. The triaxial shear test from samples obtained from this boring are plotted as figure 25.

Interpreted Shear Strength parameters are as follows:

BHS Test no. 1 - C = 520 psf, ϕ = 32.0°

BHS Test no. 2 - C = 400 psf, ϕ = 31.0°

BHS Test no. 3 - C = 350 psf, ϕ = 23.0°

Triaxial Test C = 500 psf, ϕ = 24.5°

Discussion of Results

A number of factors limit the significance of comparing BHS data obtained in this study with the triaxial data obtained. These factors include: limited number of tests; disturbed nature of "undisturbed" samples- particularly in boring location 2; the type of triaxial tests performed -

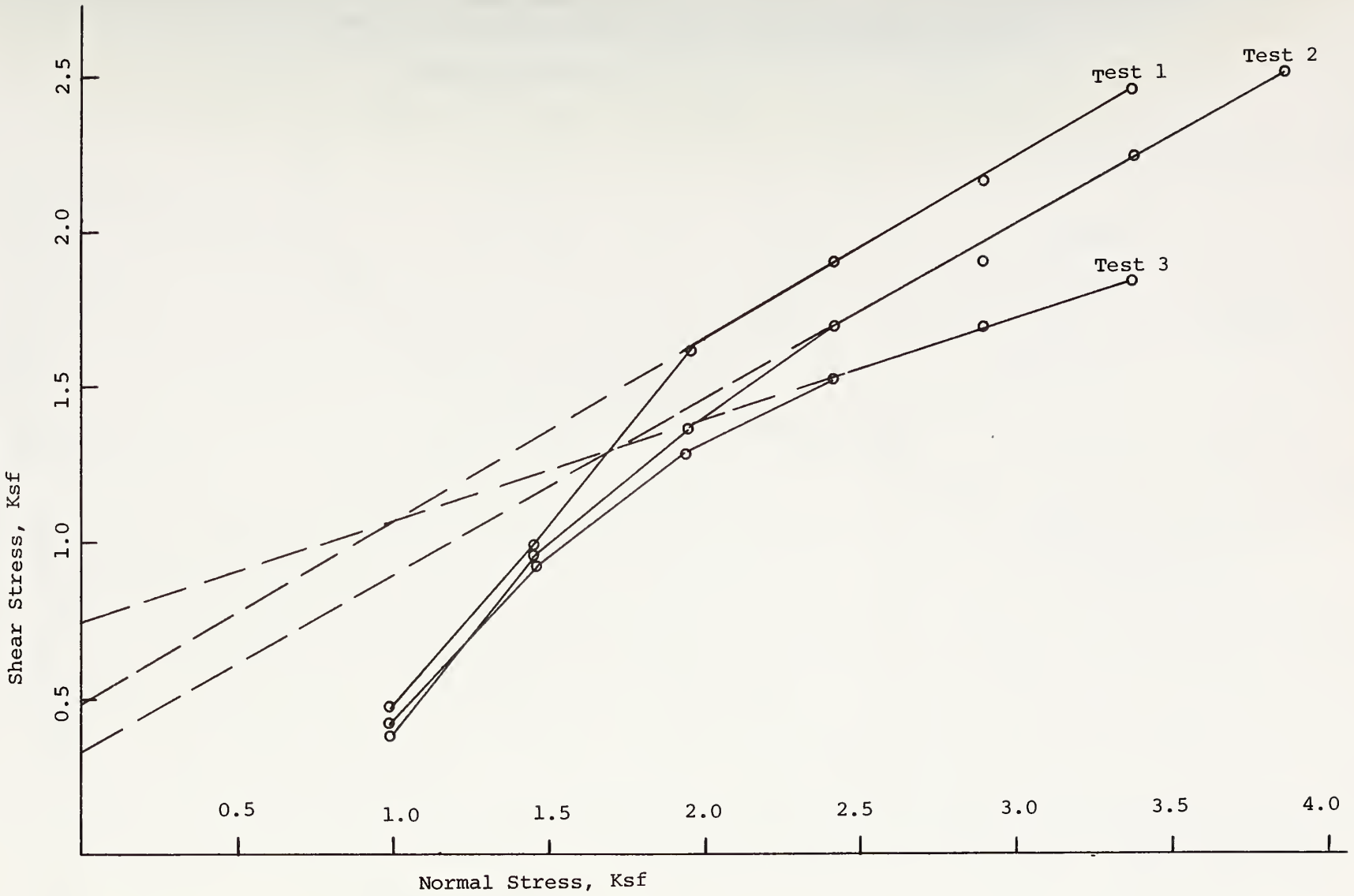


Figure 21 - BHS DATA, Rich's Computer Center
Bore Hole #2

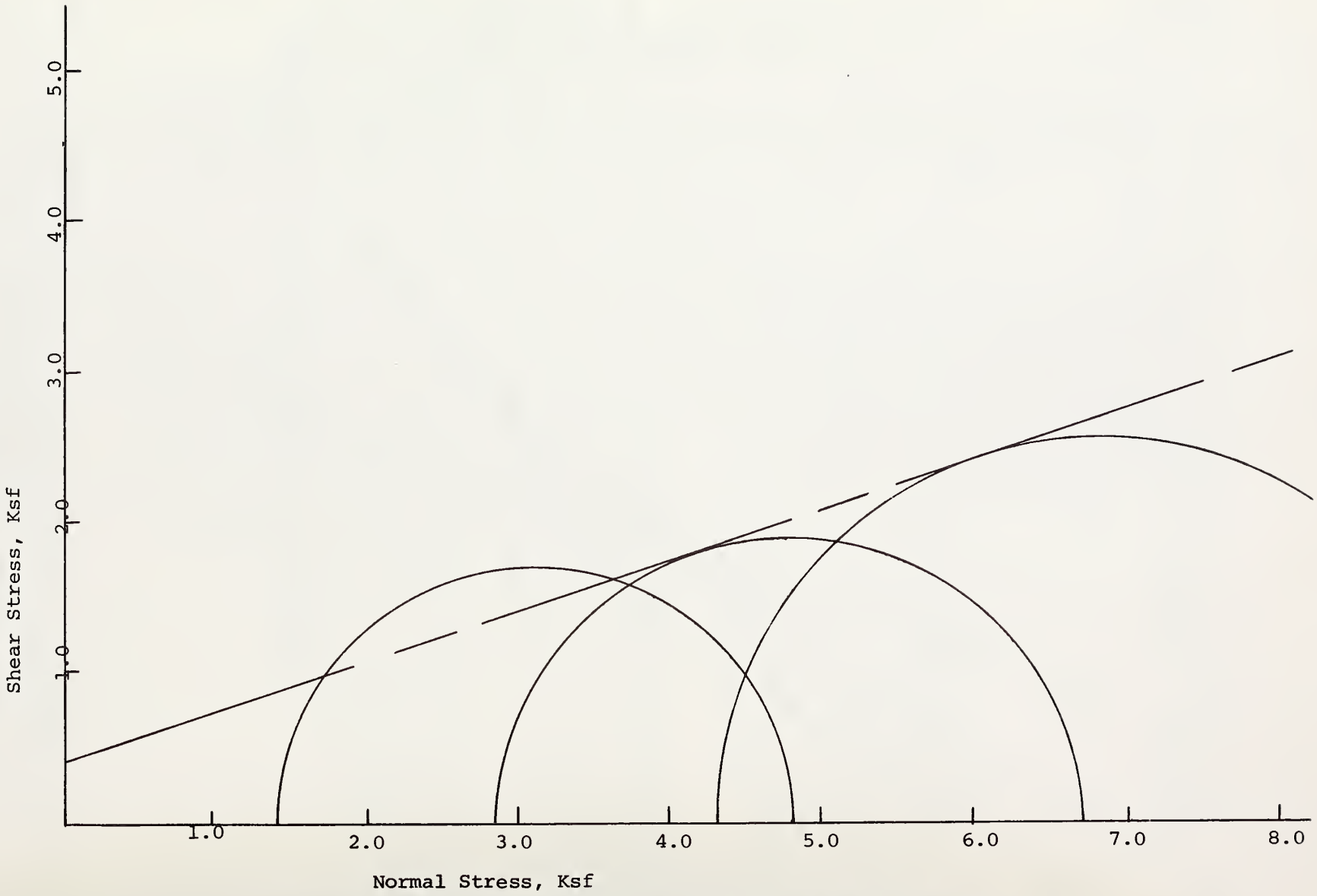


Figure 22 - Triaxial Shear Data - Rich's Computer Center
Bore Hole #2

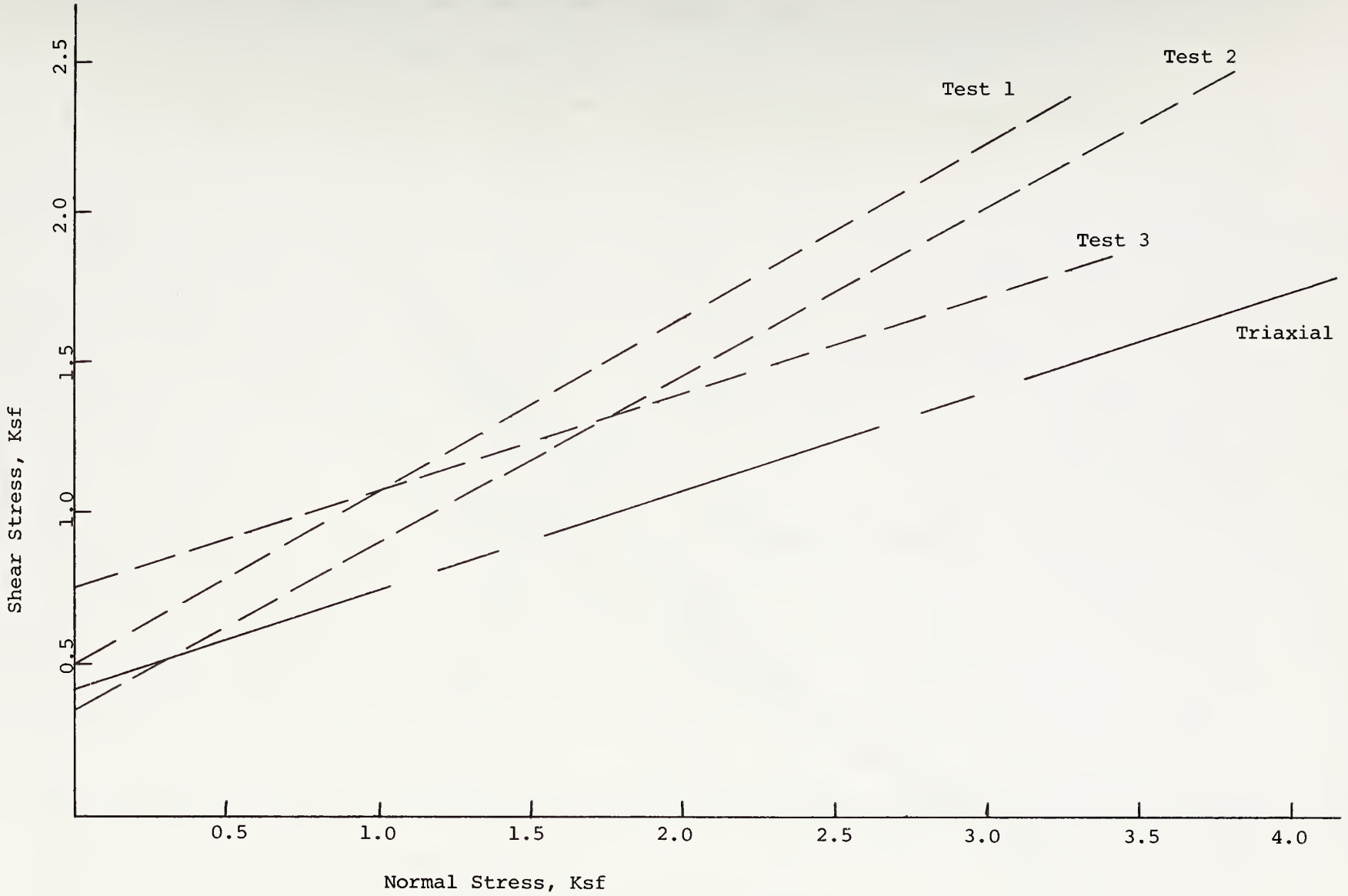


Figure 23 - BHS and Triaxial Data - Rich's Computer Center
Bore Hole #2

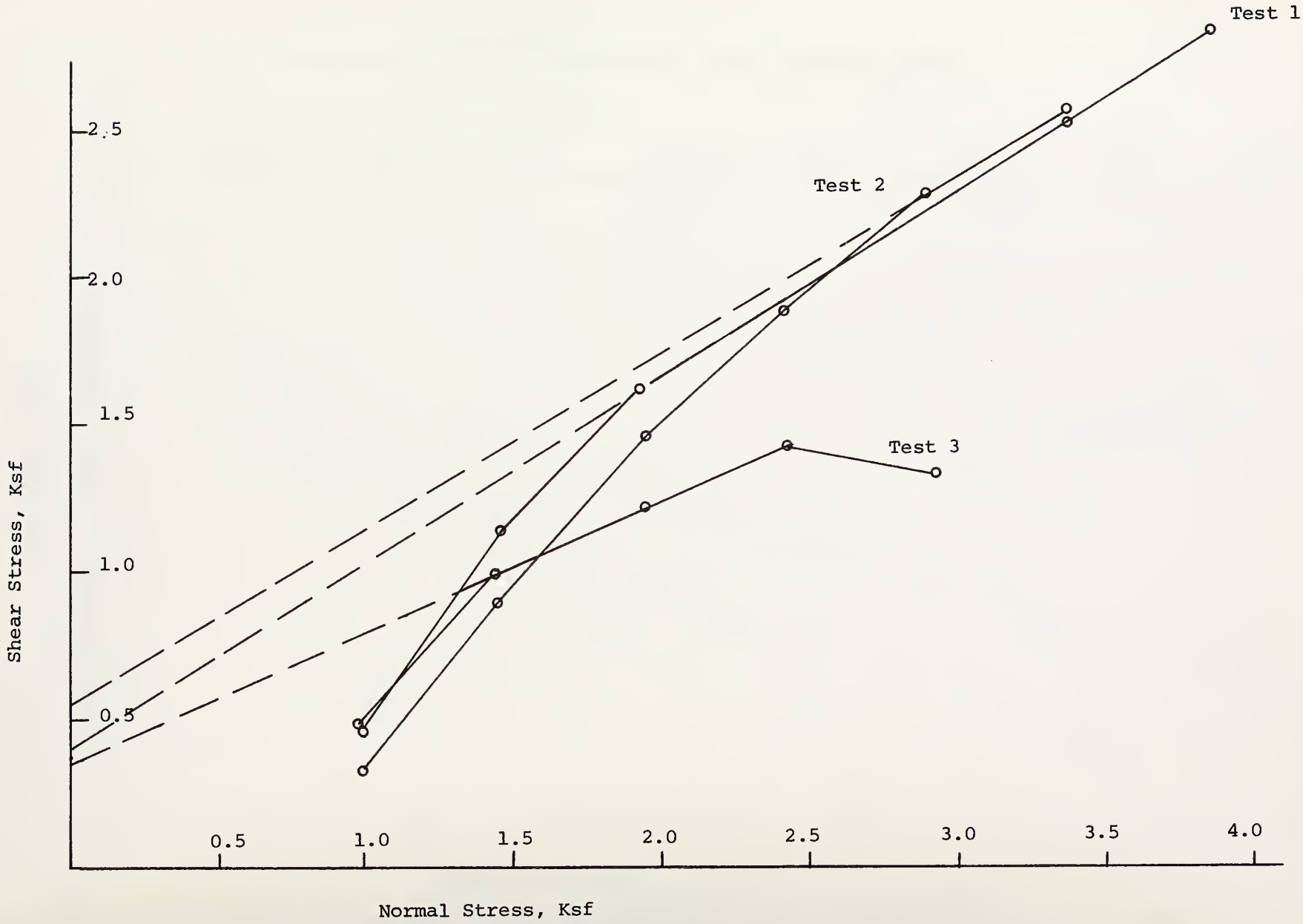


Figure 24 - BHS Data - Rich's Computer Center
Bore Hole #3

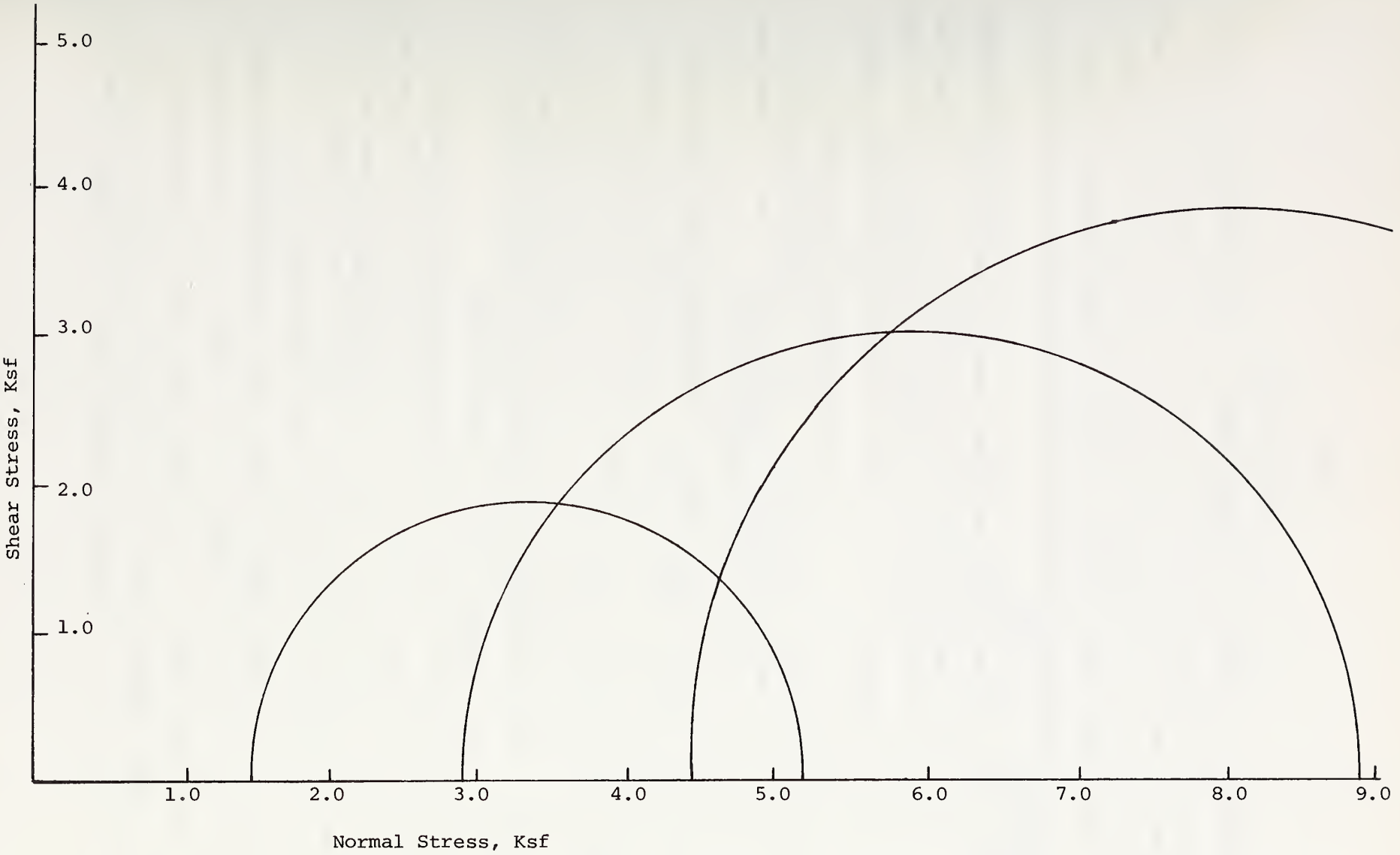


Figure 25 - Triaxial Shear Test - Rich's Computer Center
Bore Hole #3

unconsolidated, undrained while the BHS test more nearly simulates a consolidated test; early model of the BHS unit which did not have the accuracy of the commercial model; and the nature of the soils tested, which are inherently heterogenerous, with abrupt changes in engineering properties often occuring with limited depth changes. However, in spite of these admitted limiting factors, a correlation of shear strength data which could be considered surprisingly close, resulted.

Statistical evaluations of such a small sample of data can be misleading. Yet it seems appropriate to look at average values for the two test locations which contained laboratory samples in apparently good condition. The "average" shear strength parameters for the two BHS tests performed at location number 1 were: $C = 140$ psf; $\phi = 26.7°$. The average values at location number 3 for three BHS tests were: $C = 423$ psf; $\phi = 26.7°$. Differences between the averaged values and the interpreted triaxial values in the two locations were: 80 psf for C and 1.2° for ϕ, location number 1; 77 psf and 2.2° for C and ϕ, respectively in location number 3. These are probably within the accuracy that one could obtain if several complete triaxial tests were performed of "the same soil". It is unfortunate that more data were not available, and that comparisons could not be made of differences inherent in the soil itself, as could have been shown by comparing one triaxial test with another. But, for the limited time and budget involved, the test comparison was encouraging.

Another possible method of comparing test data, again with the same limitations as previously discussed, was to look at shear stress corresponding to a particular value of normal stress. Such a comparison is shown on table 3.

These comparisons indicate also, that results of BHS testing for these sites and the soils tested, appeared to compare reasonably well.

Shear Strength testing of partially weathered rock such as that tested in this study, is seldom performed because of difficulty in obtaining relatively undisturbed samples. The BHS test shows promise as a potentially faster and less expensive method of obtaining shear strength data in these hard and dense soils. Potential use in design include anchorage evaluation

TABLE 3

BHS and Triaxial Data

Rich's Site

Test #	Type Soil	Depth	BHS c (ksf)	BHS ø (°)	Triaxial c (ksf)	Triaxial ø (°)
	BHS #1 at Rich's Computer Center					
1	Partially weathered rock white, brown and tan	20"	0	31.0		
					.06	25.5
2	Granular sand and silt white and tan	14"	.280	22.5		
	BHS #2 at Rich's Computer Center					
1	Partially weathered rock white, brown and tan	19"	.480	29.5		
2	Granular sand and silt white and tan	12"	.330	29.0	.42	17.5
3	Micacaeous silt grey and gold	6"	.750	17.5		
	BHS #3 at Rich's Computer Center					
1	Partially weathered rock white, brown and tan	20"	.520	32.0		
2	Granular sand and silt white and tan	12"	.400	31.0	.50	24.9
3	Micaceous silt grey and gold	6"	.350	23.0		

Note: To convert ksf to kN/m^2, multiply by 47.9

and friction pile evaluation. The method of applying data would be to evaluate shear strength parameters C and ϕ, and use them in accepted soil mechanics calculations.

Case History No. 5: Steam Plant Site, Pensacola, Florida

In the spring of 1972, a study was made of soft, cohesive soils in a marshy area north of Pensacola, Florida.[11] The area was associated with the Crist Steam Power Plant and was being used for retaining dikes to hold fly ash and other effluents from the plant operation. The practical objective of the study was to evaluate the practicability of raising the height of dikes prior to using them for retainage of materials.

The very soft, very weak clay soils and the objectives of the study allowed field comparisons to be made of different in-situ testing means. Because of the nature of these soft clay soils, and the organic matter which was present in a portion of the subsoils, undisturbed sampling for laboratory testing was extremely difficult and costly. Yet it was felt that some laboratory data was needed as an aid in interpreting in-situ test results, particularly in evaluating the undrained shear strength of an organic, marshy soil layer which existed below the dike. Thus, the in-situ field tests could be evaluated in comparison with laboratory tests of this difficult to evaluate material.

Site and Subsurface Conditions: The Crist Steam Plant is located on the Escambia River estuary north of Pensacola, Florida. Lower marshy areas near the Escambia River and Governor's Bayou were used for retaining dikes to hold the fly ash and other effluent from plant operation. Ash Pond"A" was constructed in 1955, and has been in continuous service since that time. The dikes for Ash Pond "B" were constructed in 1967, but the pond had not yet been placed in service at the time of this study. The plant personnel hoped that the dike height could be raised prior to placing the dike into service. General conditions at Ash Pond "B" were similar to those at Ash Pond "A". Subsurface conditions in the Ash Pond "B" area prior to construction were as shown in Figure 26. The marshy

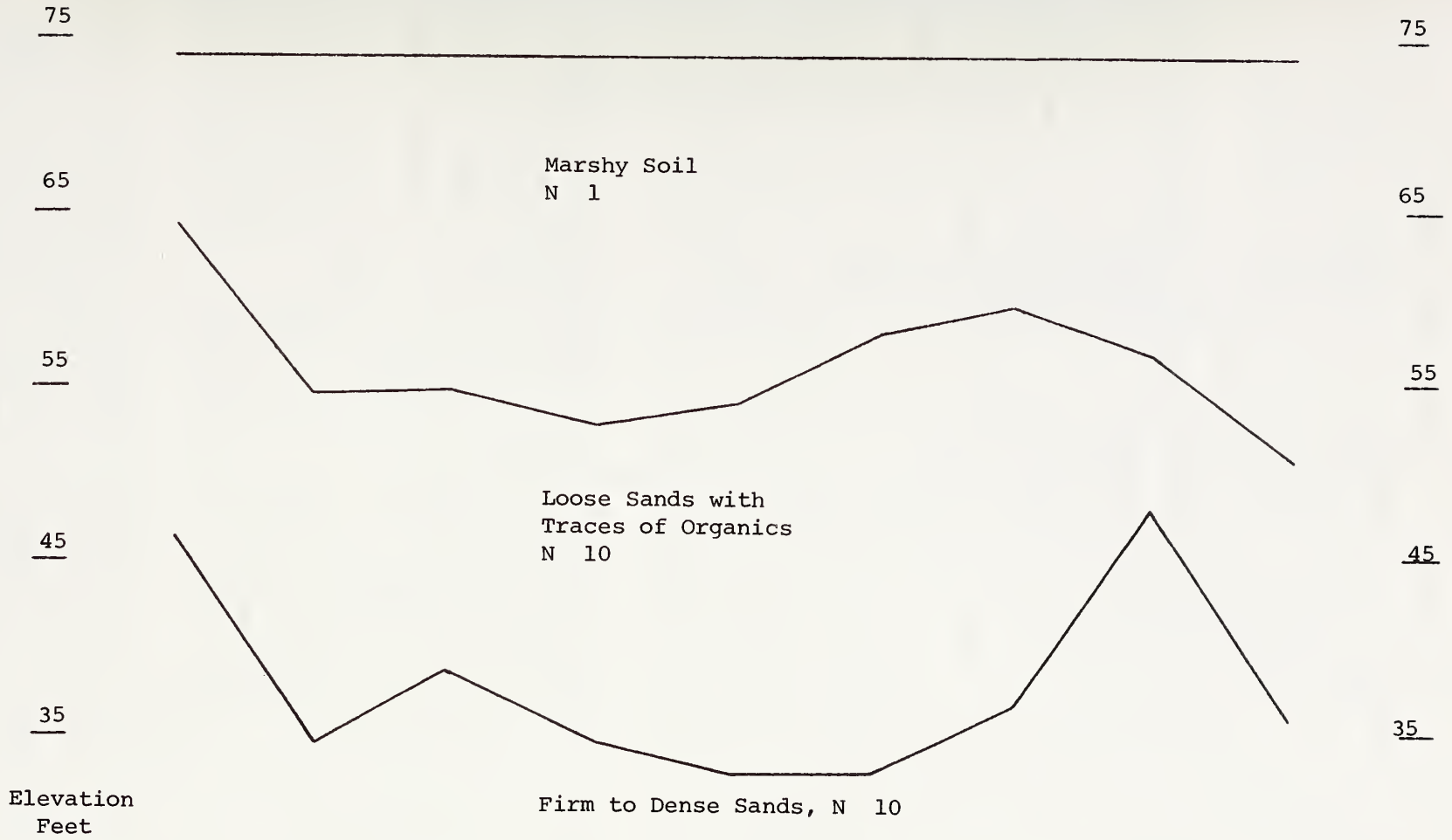


Figure 26 - General Soil Profile Before Dike (1967)
Crist Steam Plant

layer, which was a mixture of peaty clay, reeds and decayed vegetative matter, and black clay, extended to an approximate depth of 6.1 meters (20 feet). The marsh soil was underlain by very loose sands with some trace of organic matter. These loose sands increased in consistency with depth into firm to dense sands at an approximate depth of 12.2 meters (40 feet). Penetration resistances in the marsh layer were generally less than 2, indicating a very soft soil.

Figure 27, shows general subsurface conditions approximately four years after the initial dike construction in the Ash Pond "B" area. The marsh layer had compressed to an average thickness of 3.4 meters (11 feet), a reduction in height of 45 percent. In addition to compression of the marsh soil, the density of the sands immediately underlying the marsh had increased. The borings defining the subsurface profiles were made along the centerline of the dike. The decrease in marsh thickness measured were not expected to be as great at the edges of the dike. For a considerable distance beyond the edges of the dike, the existing marsh surface showed visable signs of subsoil movements. Trees were tilted away from the dike and ground waves could be observed. The average ground water level at the time of the study corresponded closely to the original ground surface, approximatley 0.3 meters (one foot) above sea level.

Figure 28 illustrates curves of the average effective overburden pressure and water content verses depth for the general subsurface conditions encountered under the center of the dike. Table 4 lists the locations and depths of undistrubed samples, Bore Hole Shear Tests, vane shear tests, and pressuremeter tests. All tests were performed within the marsh soil layer.

BHS Tests using the Bore Hole Shear device were performed in five selected borings. Undisturbed samples were taken in making the hole for these BHS tests. These samples were then used for subsequent laboratory triaxial shear strength tests. A total of five Bore Hole Shear strength tests were performed.

Vane Shear Tests were performed in general accordance with the provisiions of ASTM Specification D-2573. The vane used was 6.3 centimeter (2.5 inch) diameter by 12.7 centimeter (5 inch) high vane. NX casing was used to keep the bore hole open. Torque was applied by a hand held torque wrench. The rate of torque application was considerably greater than the rates suggested by ASTM.

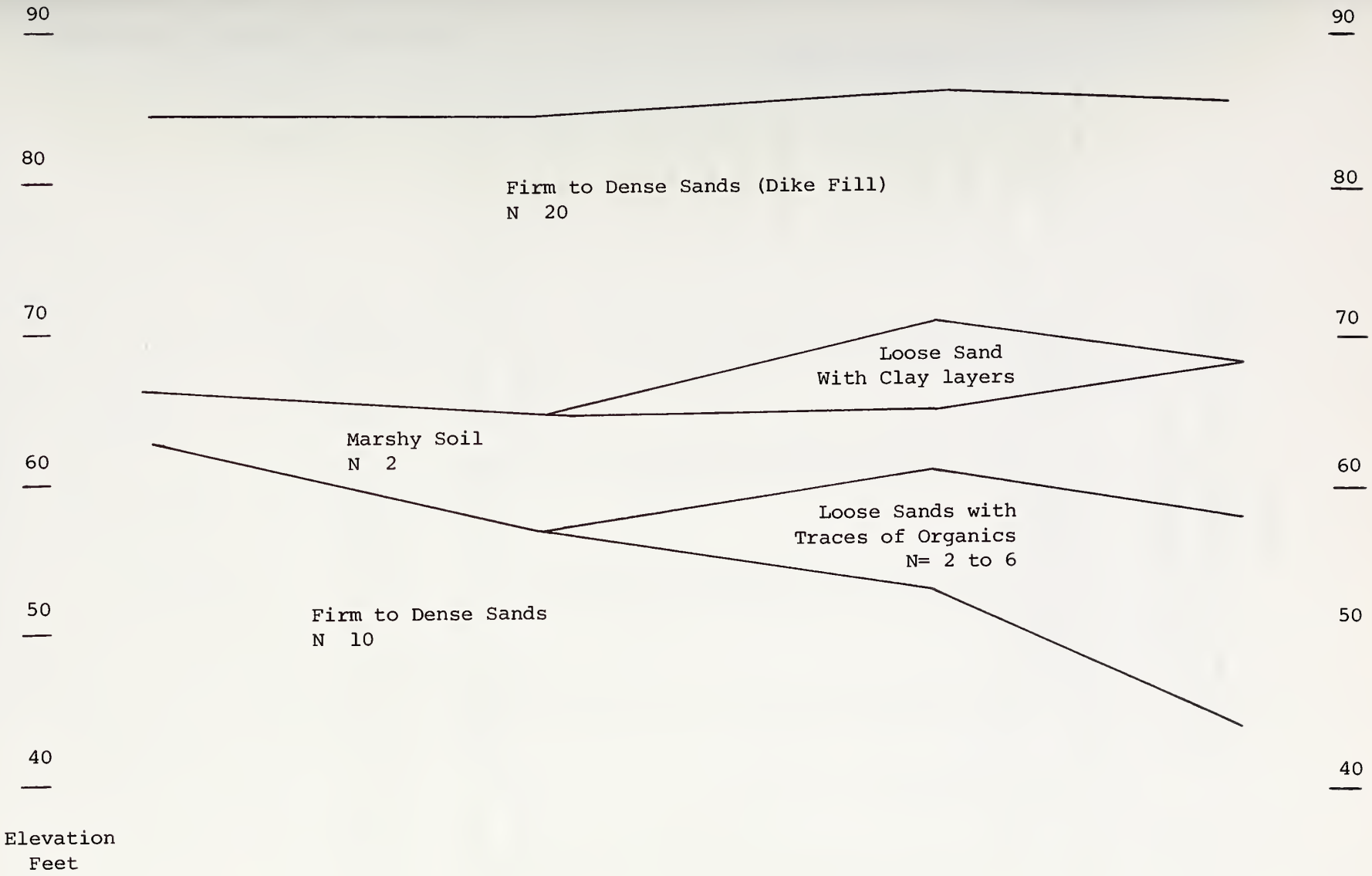


Figure 27 - General Soil Profile After Dike Consolidation, Crist Steam Plant

TABLE 4

Unit Weight and Moisture Content Data

Crist Steam Plant Site

Boring	Depth	Natural Moisture Content, Percent	Wet Unit Weight, PCF	Material Type
B-110	2.5	15	127	Dike Fill
B-112	7.5	16	122	Dike Fill
B-115	7.5	11	133	Dike Fill
B-119	15	13	131	Dike Fill
B-110	25	172	75	Marsh Soil
B-117	18	125*	102	Marsh Soil
B-118	23	95*	99	Marsh Soil
B-119	21	207*	**	Marsh Soil
B-121	25	119*	97	Marsh Soil
B-122	21	108*	**	
B-127	19	18*	118	Dike Fill
B-127	42	33*	117	Sand

*Average of moisture content at top and bottom of tube

**Sample condition prevented determination of unit weight

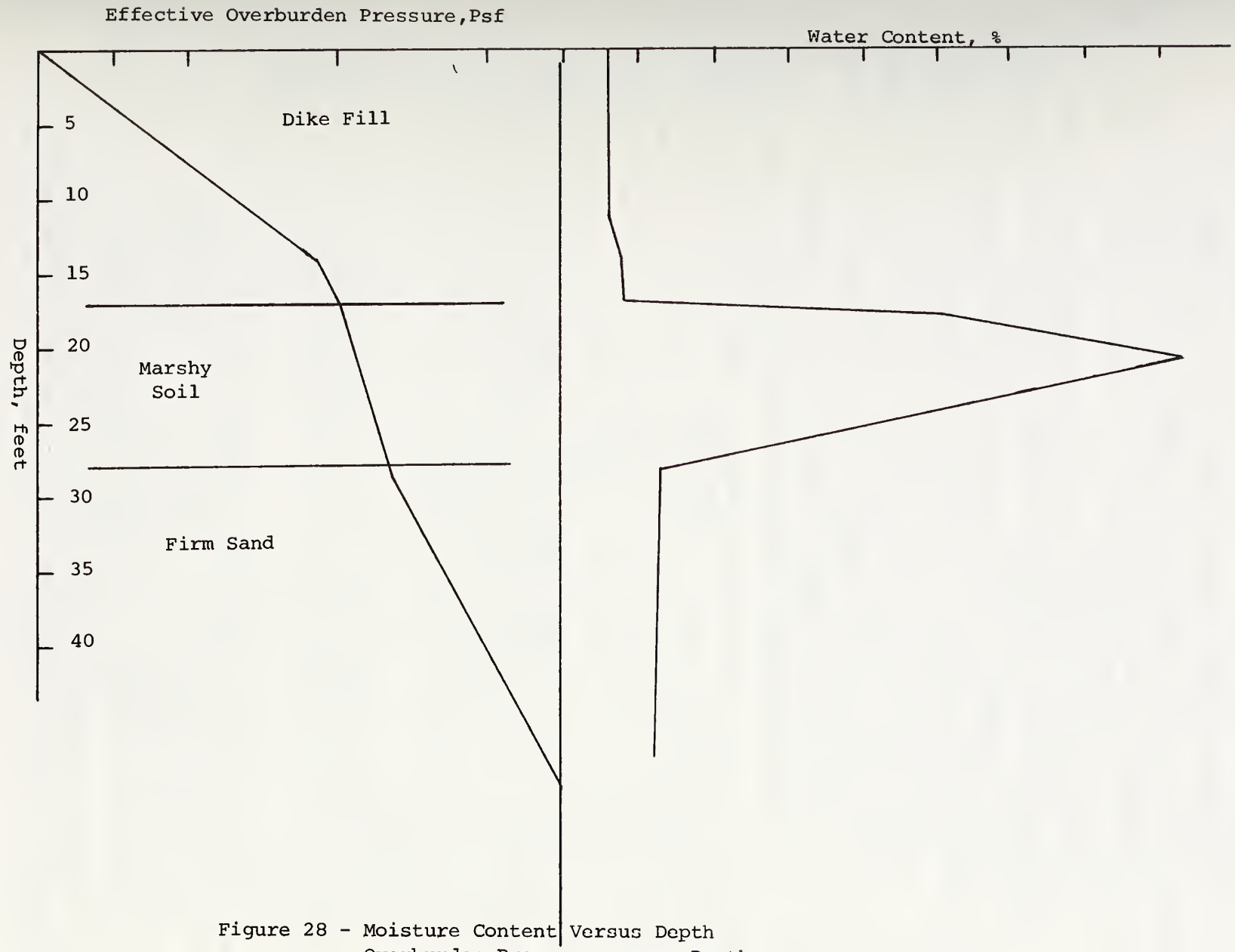


Figure 28 - Moisture Content Versus Depth
Overburden Pressure versus Depth
Crist Steam Plant

It was found that a hand application of torque in accordance with the ASTM rate (1/10 DEG/SEC) was not possible. After peak torque values were recorded, the vane was rotated rapidly through at least 10 revolutions, and a determination of remolded strength was obtained. Seven vane shear tests were performed in six borings.

Due to limited time availability of the pressuremeter equipment, only two pressuremeter tests were performed, both in one boring. No other in-situ tests were performed in that particular boring. The pressuremeter consisted of two main sections - a probe which was placed in the bore hole and a pressure-volume meter which was connected to the probe by plastic tubes through which water and gas pressure were applied. The probe was a cylindrical metal assembly with rubber membranes attached in such a manner as to form vertically aligned independent cells. The center cell applied a radial pressure to the soil around the probe while the upper and lower cells expanded simultaneously to reduce the effects of end restraint. The pressure gauge and volume meter allowed simultaneous measurement of applied pressure and resulting volume change of the probe. In operation, the pressuremeter probe was lowered into a bore hole drilled only slightly larger than the probe. At the desired depth, increments of pressure were applied to expand the center cell against the soil. For each increment of pressure, volume changes were recorded until equilibrium was reached. These data were plotted and reduced to soil strength parameters according to appropriate procedures outlined by the manufacturer of the pressuremeter.

Undisturbed samples obtained from the marsh layer were used for unit weight and moisture content determinations as well as triaxial shear strength testing. Relatively large variations were observed in the moisture contents at the tops and bottoms of individual tubes. In some cases, wood fragments were encountered in the tubes. Because the BHS tests had been made in the holes formed by the undisturbed sample tubes, it was possible to eliminate some unusually high Bore Hole Shear strength results due to the observed wood fragments in the matching undisturbed samples. Conventional unconsolidated undrained and consolidated undrained triaxial shear strength tests were performed on undisturbed samples from bore holes which had both

vane and Bore Hole Shear strength tests performed. In addition, an unconfined compression test was performed on a reasonably intact jar sample. Although several UD's were taken, the poor quality of the material resulted in significant sample disturbance and only two triaxial tests were performed on two undisturbed samples.

Results of the laboratory and in-situ tests are listed in Table 5. Results from the four borings where Bore Hole Shear tests and vane tests were performed are graphed against water content in figure 29. This figure indicates that a relatively good linear relation existed between water content and the log of undrained shear strength for the data obtained. The fact that both test types fell close to the same line was evidence that both tests produced useable results in the field.

While statical evaluation of such limited data is of limited value, the means and standard deviations for the Bore Hole Shear and vane tests are shown below:

TEST TYPE	MEAN, PSF	STANDARD DEVIATION, PSF
All	817	290
Vane	843	400
Bore Hole Shear	780	160

The Bore Hole Shear test results show significantly less variability than the vane tests, indicating a somewhat greater reproducibility. Examination of the undisturbed samples from which laboratory tests were performed for this study indicated a high degree of disturbance. This fact casts some doubt on the validity of the laboratory test results. The unconsolidated undrained tests gave reproducible results. Most field data did fall within the limits defined by the unconsolidated undrained tests and the consolidated undrained tests.

Several of the samples indicated the presence of decayed vegetation and old roots and marsh grasses. The effects of these on the vane shear strength results are difficult to quantify. It is probable that the vane values in these areas were somewhat higher than would be obtained without the presence of the reeds and other vegetative growths.

Because failures or partial failures could be observed in several locations along the dike, it was possible to estimate a value for undrained

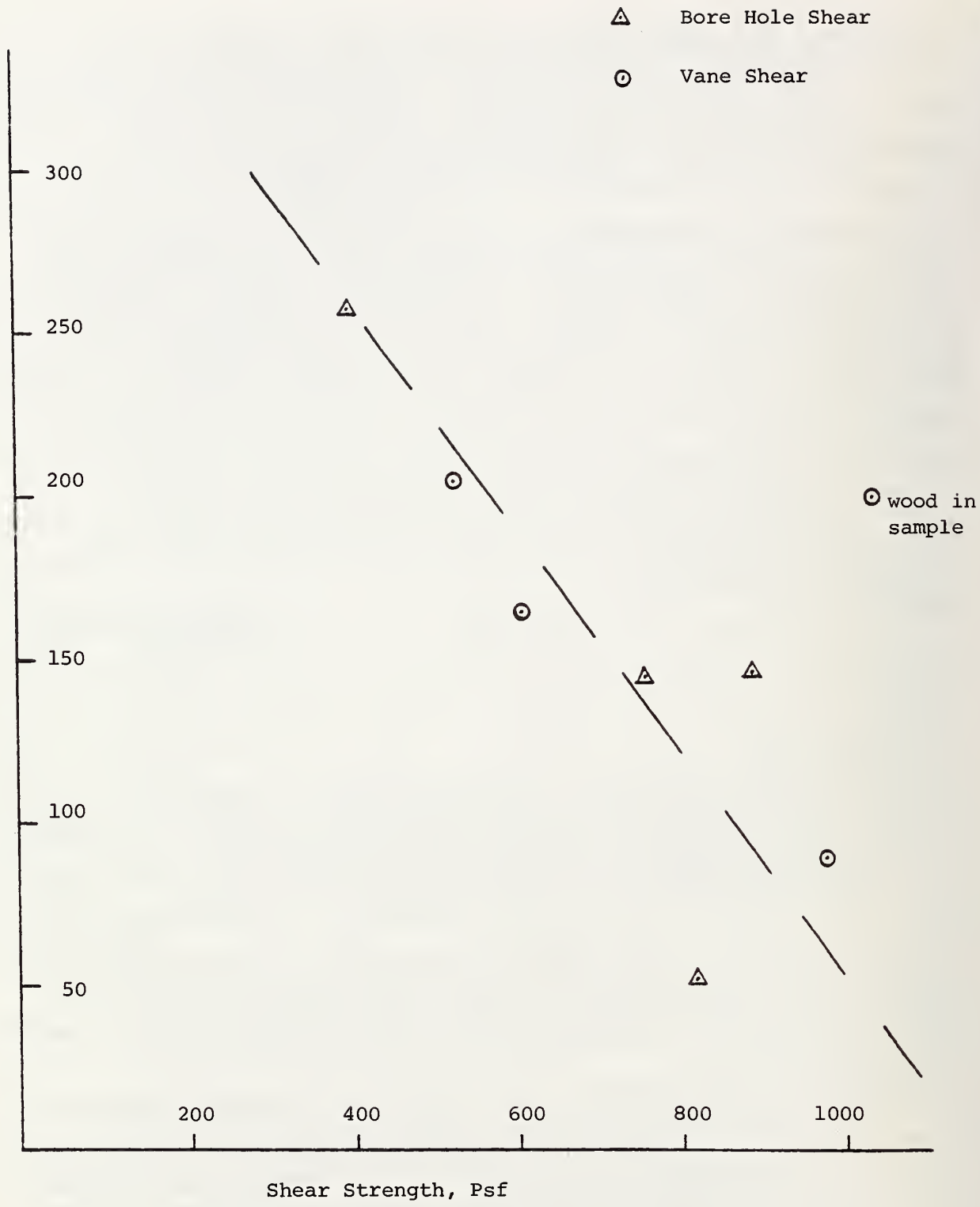


Figure 29 - Bore Hole Shear and Vane Shear Data
Crist Steam Plant

TABLE 5

In-Situ Test Results

Crist Steam Plant Site

Boring	Depth	Test Type	Su psf	Ø°	C,psf	Calc. S
B-110	25'	UNC	570			
B-110	27.5'	Vane	1360			
B-116	25.5"	Press.	1040*	10**	515	
B-116	27.5'	Press.	1200*	10**	680	
B-117	16.5'	BHS		14	374	820
B-117	18'	CU		8	480	840
B-117	18.5'	Vane	1140			
B-118	22.5'	BHS		10	403	760
B-118	23'		300			
B-118	24'	Vane	530			
B-119	18.5'	Vane	760			
B-119	20.5'	BHS		6	202	400
B-119	24.5'	Vane	610			
B-120	22'	Vane	610			
B-121	24'	BHS		20	187	890
B-121	25.5'	Vane	980			

* Pressuremeter Cohesion, C.

**Assumed friction angle

Note: To convert psf to N/m^2, multiply by 47.9

shear strength based on an assumed failure surface. For an assumed factor of safety of 1.0, the average undrained shear strength necessary to prevent failure was found to be approximately 11,975 N/m^2 (250 psf). Calculations were also made for a "squeezing" type of failure whereby soft material under the center of the dike is displaced outward without an overall failure. The undrained shear strength necessary for a factor of safety of 1.0 under these conditions was found to be approximately 19,160 N/m^2 (400 psf).

Due to the large scatter observed in the resulting data, the known presence of organics and vegetation, and the poor quality of the undisturbed samples available for laboratory testing, no definite statement can be made about the relative accuracy of the various in-situ tests used. Because both Bore Hole Shear and Vane Shear results tended to fall near the same line on an S_u vs water content plot, both tests showed meaningful results with roughly equal reliability for organic deposits. Statistical parameters on the limited data available did indicate less variability in the Bore Hole Shear tests than in the vane tests.

Because only two pressuremeter tests were performed, little could be concluded about comparative results. No water content data were available for the soil in the boring where pressuremeter tests were performed.

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BORE HOLE EARTH SETTLEMENT APPARATUS

(BESA)

Introduction

The Bore Hole Earth Settlement Apparatus (BESA) is an instrument which has not been commercially manufactured, but which holds much promise as a practicable in-situ method of predicting settlement. It has been used in research design, and limited consulting projects by the author. The design and concept is unique, in that it employs both an inflatable cylindrical membrane and separately operating pressure plates. Three different testing concepts can be applied with this instrument: similitude model load testing; elastic modulii determination; and in-situ consolidation testing. A need exists for the in-situ soil paramters that the BESA can generate, particularly the elastic modulii for use in finite element problem solving. Although additional development of the BESA concept and instrument is necessary prior to wider acceptance and commercial manufacture, the potentials of this unique instrument are readily apparent.

Synopsis

The BESA study discusses actual uses of the instrument in research design and in consulting applications for predicting settlement of building and storage tank foundations. The three separate potential uses of the BESA are described: the similitude approach which shows promise but is limited by both lack of familiarity of practicing geotechnical engineers as well as need to resolve one theoretical limitation; the elastic modulii use, which at this time offers the greatest potential of the BESA to meet a growing need; and an inside out in-situ consolidation test method. The main limitation to present use of the BESA, even in a research capacity, is lack of a commercially available instrument. Only two working instruments exist, both are in the possession of the author. Additional research is needed for developing the BESA to the point of becoming a practicable, useful and important in-situ testing method.

USES IN ENGINEERING DESIGN

The Bore Hole Earth Settlement Apparatus (BESA)[1] which is presently in its third generation, non-commercially designed stage, has been used very sparingly in applications to solving geotechnical problems. This is because the instrument is not yet commercially available, and therefore the only user to date has been the author, who has utilized the instrument primarily in a research-oriented capacity. The Bore Hole Earth Settlement Apparatus has, however, been utilized on several consulting projects in association with conventional methods. It has been used in conjunction with other methods to provide data with which to solve geotechnical problems. Two primary reasons exist for its limited use: first, the somewhat crude state of the present apparatus and resultant mechanical problems that occur during operation; and second, the lack of knowledge and experience concerning the validity of data obtained by this test method. Therefore, as of the date of this study, no practical geotechnical problem has been solved by the BESA by itself. However, a number of geotechnical problems have been solved with the aid of both BESA data and more conventional methods of predicting settlement, including consolidation theory and elastic modulus theory. Among the types of geotechnical problems to which the BESA has been applied in this manner are settlement prediction for mat foundations and settlement prediction for shallow spread footing foundations.

Soil Parameters Obtained: The Bore Hole Earth Settlement Apparatus has been used as an "inside-out" consolidation test. Load versus deformation data can be plotted in a manner somewhat similar to that performed during a laboratory consolidation test. The major differences between the BESA consolidation parameter data and that obtained in the laboratory are the horizontal versus vertical orientation, three dimensional test with the BESA versus the one dimensional characteristic of the laboratory consolidaion test, and disturbance effect differences between laboratory samples obtained from thin wall tubes versus testing of soil walls in-situ in a hole. Another significant difference is that sample "depth"

with the BESA is not defined as it is in the consolidation test. Therefore, void ratios are indeterminable without laboratory test data.

Elastic Modulii: The BESA test may be performed in a manner to obtain elastic deformation data or elastic modulii for use in elastic theory to determine settlement. This will be expanded upon in later sections of the study.

Methodology and Testing: The BESA is a cylindrical instrument which is utilized to test the soil near the walls of a drilled hole. The testing apparatus can be considered to consist of three separate parts. The first part is the cylindrical housing device to be installed in the bore hole. This housing is capable of exerting a radial membrane pressure on the walls of the hole. In addition, two sets of pneumatically operated rectangular loading plates which can be operated either simultaneously or separately, are housed within the cylindrical apparatus and inside the membrane. Measurement of the translation of the plates into the soil is made electronically, presently by strain guages. The second part of the testing apparatus is a pressure console which separately regulates the membrane pressure and the applied force on the loading plates. The third part is a strain guage indicator system to measure the plate translations. See figure 30.

Test procedures for determining in-situ consolidation, elastic modulii, and similitude model load parameters are outlined below.

BESA consolidation tests:

(1) The first step in the performance of this test is to drill a hole in the soil by using a 7.6 centimeter (3 inch) diameter machine-driven or hand operated auger. The bore hole diameter is critical, but can vary from approximately 7.6 to 8.9 centimeters (3.0 to 3.5 inches) within the area to be tested.

(2) After the hole has been drilled (or formed by the insertion and removal of a thin walled tube), the Bore Hole Earth Settlement Apparatus is inserted to the desired test depth.

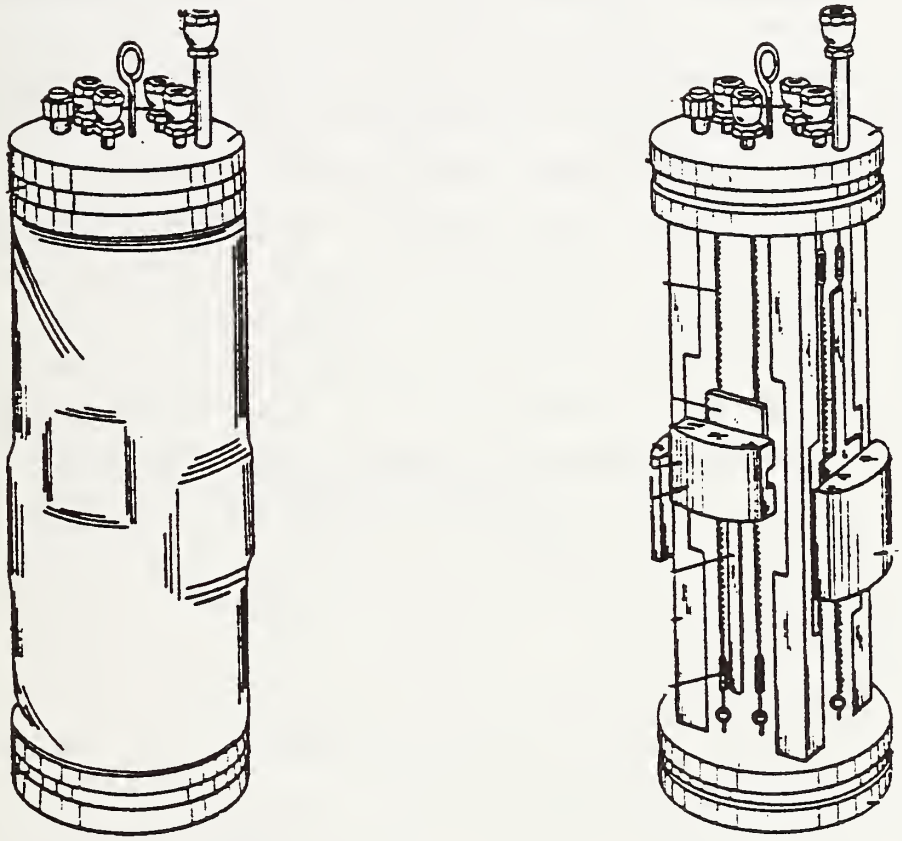


Figure 30 - BESA Apparatus
2nd Generation

(3) The membrane is expanded to a predetermined expansion pressure. This pressure is normally computed to approximate in-situ overburden stresses, but may be increased if desired, to simulate changed conditions future additional predicted surcharge.

(4) One set of model plates is then expanded pneumatically with a predetermined loading force.

(5) The first load can either be applied for a predetermined time period, or more practically, until consolidation has virtually ceased. For sands and non-clayey silts, this time interval is normally short and usually less than ten minutes. Movement is determined by the strain guage indicator. A load versus movement reading is taken.

(6) Additional load versus movement readings are taken as desired. Load increments are increased according to predetermined test procedures.

(7) Interpretation of load versus movement data obtained has not yet been standardized. Void ratio data, if required, must be obtained by laboratory testing. This can be accomplished using the undisturbed samples obtained by thin wall tubes used to form the hole.

BESA elastic modulii test:

(1) Steps one, two and three are identical to those in the consolidation sequence above.

(4) One set of model plates is expanded pneumatically with a predetermined load intensity.

(5) The additional load increments can be applied as desired.

(6) The slope of the linear portion of the load versus movement curve is related to the elasticity modulus. Refinements of this test includes varying the membrane pressure. An example calculation is presented below for use of the BESA to obtain elastic modulii for settlement predictions.

G. T. McDaniel[2] in his use of the BESA to predict settlement of different foundation types, made an analysis of BESA data to develop a semi-empirical method of arriving at confining stress, σ_c corresponding to modulii of elasticity. He plotted membrane pressure versus bore hole radius deflection and projected the initial slope between the first two pressure increments back to zero pressure. He then read the deflection corresponding to the initial bore hold radius. This zero deflection was subtracted from each deflection increment to provide the actual change in bore hole radius for each membrane pressure increment.

The modulus of elasticity was calculated by both the tangent and the secant method. The tangent method, E_r, gave the modulus of elasticity between two pressure increments. It was computed by the following formula:

$$E_T = \frac{(1-u)\Delta P_n \, r_{n-1}}{r_n - r_{n-1}}$$

where E_T = Tangent modulus of elasticity between increments n-1 and n

u = Poisson ratio (=0.35)

P = Change in membrane pressure from increment n-1 to increment n

r_{n-1} = radius of bore hole at end of increment n-1 before application of pressure p_n.

r_n = radius of bore hole after the application of p_n.

The tangent modulus of elasticity was higher than the secant modulus. It described the modulus of elasticity based on the original bore hole radius, total pressure membrane and total bore hole deformation and was obtained as follows:

$$E_s = \frac{(1 + u)\, p_n\, r_o}{r_n}$$

where E_s = secant modulus of elasticity at membrane pressure p_n

u = Poisson's ratio ($u = 0.35$)

p_n = membrane pressure at increment n

r_o = initial bore hole radius

r_n = bore hole radius after membrane pressure p_n has been applied

Although the E_T value was higher than the E_S, McDaniel found that it was also more erratic, with its values increasing, as would be expected, with increasing membrane pressures. The secant method tended to average out the results and give a smoother data plot.

The actual field modulii had to be plotted versus corresponding continuing pressure in order to be used in the layer strain elastic analysis. The following general relationship was developed by McDaniel to compute confining pressure σ_c: $\sigma_c = 2/3\ \sigma_{o'} + \frac{N \sigma_m}{20}$

where: σ_c = confining pressure on the soil deformed around the bore hole

$\sigma_{o'}$ = effective overburden pressume

σ_m = membrane pressure

N = standard penetration resistance

BESA similitude model load test:

(1) Steps one and two are similar to the above tests.

(3) The membrane is expanded to a predetermined pressure. This pressure consists of two components: a surcharge pressure which is similar to that determined for the above two cases; and a new component, a "simulated gravitational pressure" to compensate for size effect distortions.

(4) Steps 4 and 5 are similar to the above procedures, with the exception that the larger set of model plates must be expanded first.

(6) At the completion of load versus movement readings for the largest model plate set, this set is retracted into the housing.

(7) Membrane pressure is increased in accordance with distorted similitude theory because of a larger "simulated gravitational pressure" component. The calculation of the "SGP" is beyond the scope of this study.

(8) The smaller plate set is then expanded. Load versus movement readings are as previously described.

(9) Results are interpreted by relating settlement intensities and ratios, and applying distorted similitude techniques.

Settlement parameters using BESA similitude model load test methods have been successfully obtained in Aolian sands, Loess silts and sandy silts, and Piedmont residual micaceous sandy silts and silty sands.[3] Data have also been obtained through similitude model load testing in alluvial clays.

Use of the BESA method for determining consolidation parameters has been extremely limited. Direct comparisons with laboratory consolidation testing are expected to be different from the laboratory test results because of the three dimensional characteristic of the BESA test versus the two dimensional characteristic of laboratory consolidation tests. Additionally, the "depth" of sample is indeterminable with the field BESA method, while it is measurable with the consolidation test. Another source of difference could be the vertical orientation normally involved in laboratory tests versus the horizontal orientation involved with the BESA tests. However, independent research indicates that this difference may be minimal.(4) The testing to date has been too limited in order to evaluate a degree of accuracy for use of the BESA to provide consolidation parameters.

The degree of accuracy for use of the BESA to provide elastic modulus as determined by several tests performed within the Piedmont geologic region of South Carolina and Georgia, is very promising. As the soil is tested in its natural stress and water regime, the in-situ modulus of elasticity may potentially be more accurately determined by this test method than by laboratory test methods. Results of predicting well-documented settlement case histories of a mat foundation and a spread footing foundation, as well as predictions of settlement of a plate bearing test, show very reasonable accuracies. Examples are presented in the "Case Histories" section of this study.

Insufficient test data exist in order to quantify a standard error or a percent error one can anticipate in using BESA elastic modulii data. However, for residual soils, BESA data appear significantly more accurate than consolidation data for settlement prediction.

Predictions of settlement of a prototype load test by utilizing results of the similitude model load test provided by the BESA show good accuracy in predicting settlements with relatively non-cohesive soils, and poor results with cohesive clays. Statistical errors in predicting prototype footing settlement for Aolian sands produced a 14% average error and an 11% average error in Loess silt. A 157% average error resulted in the prediction of settlement of alluvial clays utilizing the similitude model load test procedure. More specific information is provided in a later section of this study.

Although the documented accuracy of the Bore Hole Earth Settlement Apparatus is meager and is based on very limited usage, the expected accuracy of this test method in performing both in-situ consolidation tests and in-situ elastic modulii tests is potentially excellent. A significant limitation exists with the use of the Bore Hole Earth Settlement Apparatus in the similitude model load test method. This limitation is a difficulty in developing a theory which can be readily understood and accepted by experts in the field of soil mechanics. The distorted similitude theory has been developed to a considerable degree by the author. However few soil engineers are knowledgeable in the art of distorted similitude analysis. Furthermore, a practical limitation exists in determining the amount of "simulated gravational pressure" to apply from the theory. However, some potential does exist for this limitation to be reduced to an acceptable level. No such theoretical limitations exist with the in-situ consolidation test method or the in-situ elastic modulii test method in using the BESA.

Limitations Imposed by Site Conditions: Problems have been encountered in certain soils due to cave-ins of the bore hole caused by ground water or weak and compressible soils. In many cases this problem can be circumvented by testing immediately after a hole has been advanced, or through the use of drilling mud. Testing can also be limited by bearing capacity failure of the soil in exceptionally weak soils. Another potential limitation is the length of time required to perform in-situ consolidation tests in clayey soils.

Soil Properties and Empirical Relationships: Scil properties used in design derived from data provided by the BESA in-situ consolidation test method are similar to those obtained in the laboratory consolidation test, differing only in that void ratios will not be determinable from this test without supplemental laboratory information. Void ratios, if necessary, will have to be determined by additional laboratory testing of samples obtained from the same soil.

Soil properties and empirical relationships derived from in-situ elastic modulii testing with the BESA soil properties and empirical relationships should be similar to those obtained in the laboratory cyclic triaxial test. These data can be used for settlement evaluation, deformation prediction, and to provide parameters for use in finite element studies.

The distortion factors associated in extrapolating model load tests with the BESA to a prototype test can potentially be used for design of shallow spread footings and possibly for mat design.

IN-HOUSE RESEARCH AND DEVELOPMENT

The in-house research and development conducted for the Bore Hole Earth Settlement Apparatus is also considered to be that research and development performed under the direction of and/or by the author. A portion of this research and development was performed at Iowa State University in the partial fulfillment of a Ph.D. degree. Another phase was performed by a graduate student at the Georgia Institute of Technology under the direction of the author and Dr. Richard D. Barksdale, Professor of Civil Engineering. In this latter case, the research and development efforts were utilized to predict documented case histories of settlement. A number of the in-house research and development tests included in this summary are further expanded upon in the Case Histories section of this paper. The BESA has not yet been utilized by itself to provide design data for on-going consulting projects. The in-house research and development involved a number of tests performed in 1966 during the initial research conducted at Iowa State University and a number of tests performed in 1974 with documented case histories of settlement of structures.

The primary objective of the initial research conducted for the Bore Hole Earth Settlement Apparatus was to utilize the BESA concept to reduce distortions present in model load testing procedures to predict prototype settlement response.[3] Field tests were conducted in three general soil types: a uniformly grained fine sand; a Loess silt; and an alluvial clay. Unified classification of the three soils were SM, ML, and CH, respectively. A total of fifty (50) complete in-situ load

settlement tests were performed during this research phase. The two basic test methods employed were an unmodified plate load type of test utilizing the pressure plates of the BESA device without applying membrane pressure, and the "similitude" model load test employing both surcharge pressure components and a "simulated gravitational pressure" component.

The sand tests were conducted in Story County, Iowa in a fine sand classified by the U.S. Soil Conservation Service as Thurman loamy fine sand. This sand was chosen because of its uniform nature, its very low cohesion, and the relatively large amount of information that had previously been obtained from it. A total of 22 BESA tests were performed, 12 utilizing membrane pressure and 10 tests without membrane pressure. These 22 tests included 11 sets of test data with the smaller model footing and 11 sets with the model footing which was 1.5 times larger. At least two different sized model footings are required in order to evaluate model/prototype relationships and predict settlements from model tests. In addition, one prototype footing consisting of a 38.1 by 50.8 centimeter (15 by 20-inch) plate load test was performed.

Two objectives were involved in the evaluation of the significance of the BESA test results. The first was the comparison and evaluation of the ratio of larger model plate settlement divided by smaller model plate settlement at the same applied load intensity in relation to the theoretical ratio of settlements, and the second was the prediction of prototype plate load settlement from data obtained and extrapolated from the model load tests.

The loess soil tests were performed in a cut of Wisconsin age loess, a wind deposited silt known for its ability to stand in deep faces. A total of 18 BESA tests were performed in loess; 8 tests were performed with the application of membrane pressure and 10 tests were performed without membrane pressure. One prototype test was performed with the 38.1 by 50.8 centimeter (15 by 20-inch) plate in loess.

The clay soil tests were performed in allvuial clay located within the Missouri River floodplain. A total of 10 BESA tests were performed, 8 with the utilization of membrane pressure, and two with no membrane pressure.

Results of this research supported the similitude model load test theory. Tests performed with membrane pressures approximating additional gravitational force resulted in prototype/model settlement ratios close to the theoretical value of 1.50. For the 12 tests performed with membrane pressure in sand, the mean value of the similitude model load test was 1.53; for the eight tests performed with membrane pressure in loess, the mean value of the tests was 1.56; for the eight tests performed with membrane pressure in clay, the mean value was 1.57. On the other hand, these ratios for the 22 tests performed without membrane pressure were as follows: in sand, a mean value of 1.19; in silt, a mean value of 0.89; in clay, a mean value of 1.22. A conclusion reached from this research if not proven, at least strongly reinforced by these test results, was that the amount of distortion involved in the settlement phenomena due to size effects was significantly reduced in the sand, silt, and clay soils tested when the SGP was applied.

In comparing results of utilizing the similitude model load test data and ratios obtained to predict larger prototype "footing" response, results were as follows: in sand, the error in predicting actual prototype footing settlement was an average of 14%. In loess, the error in predicting the actual prototype footing settlement was an average of 11%. In alluvial clay, the error in predicting prototype footing settlement was an average of 157%. The conclusion reached from this summary is that potential exists for BESA similitude testing in predicting prototype footing response for non-clayey soils utilizing this concept and test procedure.

In the intervening period between the middle 1960's and 1974, the Bore Hole Earth Settlement Apparatus was redesigned in order to produce a more compact instrument which would fit within a 7.6 to 8.9 centimeter (3.0 to 3.5 inch) diameter bore hole. The general test method was subsequently modified to use both conventional consolidation theory and elastic theory rather than similitude model load test theory because of difficulties in developing a completely acceptable theoretical justification for the distorted similitude approach, as well as because of general unfamiliarity of similitude analysis by the soil engineering profession. Another factor in this decision was the apparent limitation in the use of the BESA similitude concept to accurately predict prototype response in clayey soils.

CASE HISTORIES

1. Plate Bearing Test: A plate bearing test was used as part of a research project by G. T. McDaniel[2] as a means of correlating data obtained with the BESA, and also as a means to become familiar with operating characteristiccs of the device. Modulii of elasticity were determined by the BESA and these were used to predict settlement of the prototype bearing plate. Additionally, the BESA was used to perform a radial in-situ bore hole plate load test. In this test the membrane was inflated to the at rest overburden pressure (K_o) to reconsolidate the walls of the bore hole, and the settlement plates were extended in increments to obtain a load versus "settlement" curve. The resulting curve is shown in figure 31.

From this curve, the slope of the initial straight line portion of the graph was K= 0.000917 cm/kn/m^2 (0.002488 in/psi). To evaluate the load-deflection curve for the Bore Hole Plate Load Test, the following formula was used:

$$S = P_F\ (K)\ \left(\frac{bf}{bp}\right)^N \qquad (5)$$

where: S = settlement of foundation of width bF

P_F= pressure on foundation bF

K = slope of load/deflection curve inches/psi

bF = width of foundation (plate load test width = 30.5 centimeters (12 inches)

bp = width of plate in BESA (b = 5.44 centimeters (2.14 inches)

N = correlation factor varying from 0.5 to 0.7

Thus, substituting and using N = 0.7:

$$S = (12.5)\ (0.002488)\ \left(\frac{12}{2.14}\right)^{.7} = 1.04 \text{ inches}$$

For N = 0.5:

$$S = (12.5)\ (0.002488)\ \left(\frac{12}{2.14}\right)^{.5} = 0.073 \text{ inches}$$

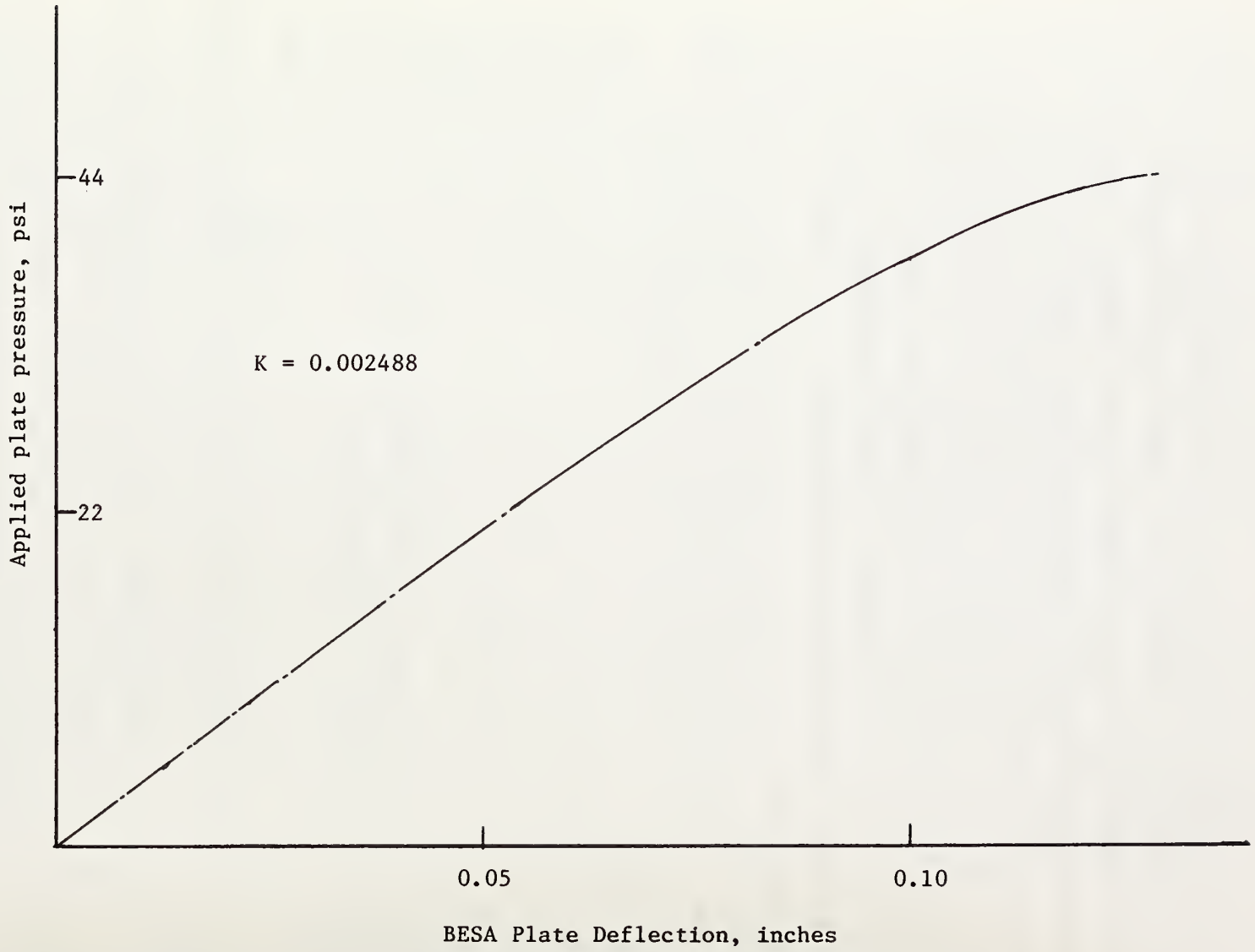


Figure 31 - Load-Deflection Curve for Bore Hole Plate Load Test

The field plate load test resulted in a deflection of 2.62 centimeters (1.03 inches) corresponding to a load of 86.1 kN/m^2 (12.5 psi). Thus, a correlation factor of 0.7 results in very close agreement. A conclusion which can be reached, is that the BESA Plate Load Test results are reasonable, and for a correlation factor of 0.7, were extremely close to actual field plate load test results.

The BESA was also used to determine the in-situ modulus of elasticity of the soil tested. Horizontal and vertical thin-walled tube samples were obtained. Reload modulii, Er were determined with the use of triaxial apparatus. Figure 32 shows results of triaxial modulii and BESA modulii versus confining pressures.

The BESA modulus data were then used to compute or predict the settlement of the Plate Bearing Test. The soil below the plate was divided into six layers of 30.5 centimeter (12 inches) thickness each, and the increase in stress due to the loading of the plate was computed for the midpoint of each layer using Westergaard stress distribution influence factors. The overburden stress was calculated and the lateral confining stress was estimated using Ko = 0.5. The secant modulus Es from the BESA tests which most nearly approximated the vertical laboratory modulus was selected. Results are summarized in Table 5.

From the elastic modulii calculations above, it is seen that the calculated settlement of 0.257 centimeters (0.101 inches) is very close to the measured settlement of 0.262 centimeters (0.103 inches). The fact that 3 different modulii were measured during 3 different testing efforts, and the modulus which most closely approximated the laboratory modulus was chosen, must be recognized. Again, a conclusion can be reached that modulus results using the BESA were reasonable, and in fact, settlement predictions using the closest of three sets of data, were very close to actual measured settlements. The significance of the very high accuracy of the BESA in predicting plate load test results using elastic modulii in the manner performed at this test site is reduced by the fact that the "best of three" sets of data was used.

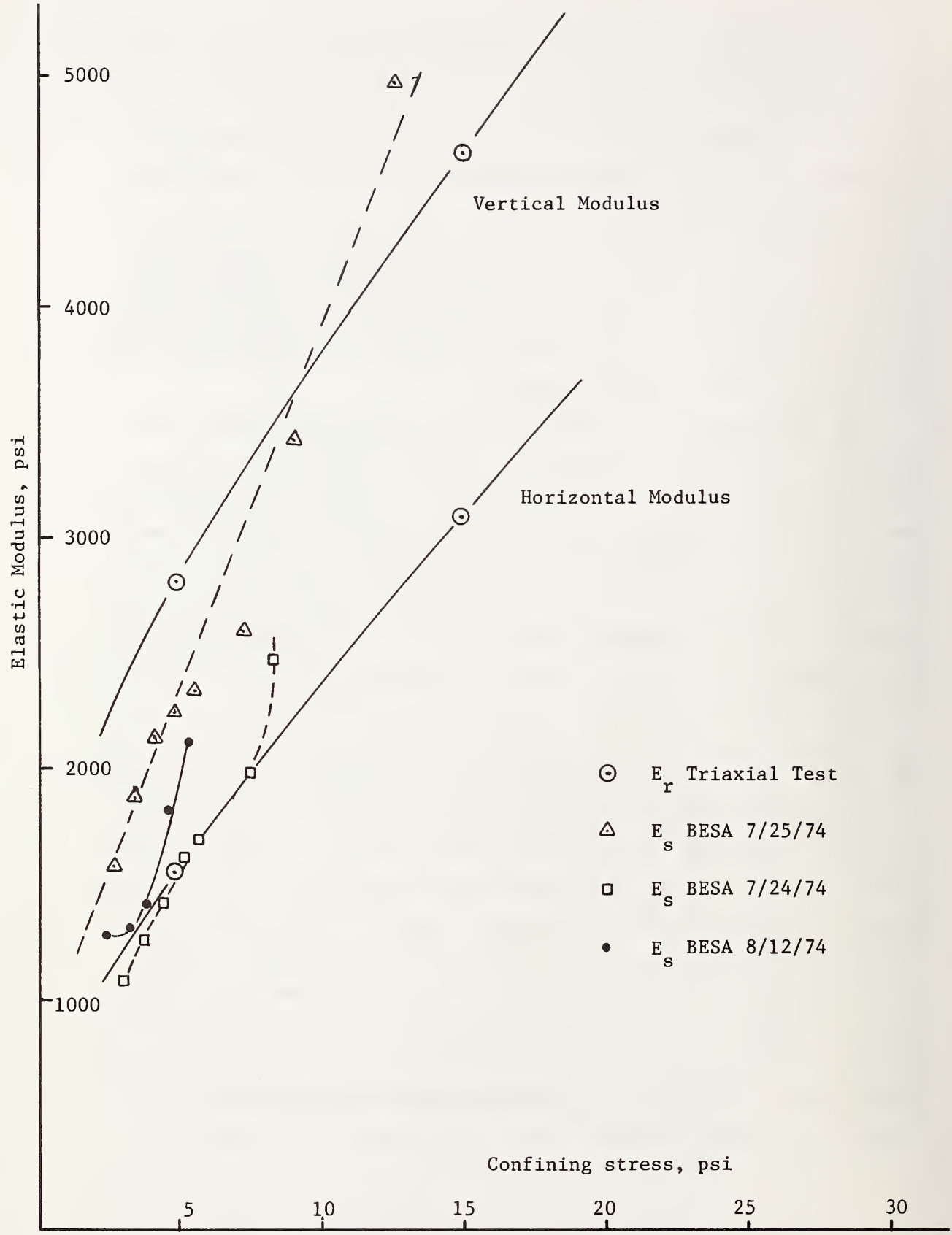


Figure 32 - Plate Load Test Modulus Data

TABLE 6

Plate Load Test Data and Calculations
using BESA Elastic Modulus Data
and Layer Strain Analysis

Pressure 12.5 Psi (86.12 kN/m^2)

Layer	H inches	Depth to Mid Layer inches	σo Psi	Iw	$\Delta \sigma v$ Psi	$\sigma v = \sigma o + \Delta \sigma v$ Psi	$\sigma_n = 0.5 \sigma v$ Psi	E_s Psi	$\Delta H = \frac{\Delta \sigma v H}{E}$ inches
1	6	3	.18	.70	8.75	9.45	4.77	1575	.0333
2	6	9	.56	.30	3.75	6.06	2.16	900	.0250
3	6	15	.94	.14	1.75	2.69	1.34	675	.0155
4	12	24	1.50	.07	.87	2.37	1.18	625	.0167
5	12	36	2.25	.035	.45	2.70	1.35	675	.0080
6	12	48	3.00	.02	.21	3.21	1.60	750	.0034
								Total Settlement	.101 in.

Measured Settlement 0.103 in.

Calculated Settlement 0.101 in.

Note: To convert psi to kN/m^2, multiply by 6.89.

To convert inches to centimeters, multiply by 2.54.

2. Spread Footings: In 1952, the U. S. Army Corps of Engineers conducted an investigation near Greenville, South Carolina at Donaldson Air Force Base (presently called Donaldson Center).[6] This investigation was to determine the behavior of soils under applied loads of a proposed aircraft hangar facility. A relatively large concrete mat was constructed and load-tested with lead weights. The mat was 5.48 meters (18 feet) by 5.48 meters in plan dimensions, and 9.1 meters (3 feet) thick. The mat was constructed in an excavation 1.98 meters (6.5 feet) below the existing ground surface.

The soils underlying the test mat, and within the area of the proposed hangar facility, were firm sandy micaceous silts and loose silty micaceous sands with standard penetration resistances varying from 6 to 9 blows per foot. The ground water table existed approximately 2.1 meters (7 feet) below the ground surface.

The test mat was loaded in five equal increments to a maximum load equivalent to 1543 pounds per square foot (73,909 N/m^2) using lead weights. Each increment of load was maintained for a period of approximately 24 hours. Settlement readings were taken at each corner and a number of referenced bench marks. The settlement of the foundation was found to almost instantaneously follow load application.

Consolidation testing of undisturbed samples was performed by the Corps of Engineers. With these data, the Corps predicted settlements for each load increment. The settlements predicted were significantly higher than actual measured settlements that occurred subsequently. Total settlement corresponding to maximum loading of 250 tons (1543 pounds per square foot) was calculated to be 12.67 centimeters (4.99 inches).

During and after construction of the hangar facility, the settlement of each hangar footing was monitored. These measured settlements along with the test mat measured settlements were evaluated with settlement predictions using layer strain analysis and modulii of elasticity determined from both triaxial testing and by the BESA test method.

A soil test boring was advanced adjacent to the hangar foundation site. Standard Penetration Tests were performed at 0.63 meter intervals (2.5 feet). Undistrubed thin wall tube samples were taken at depths of 1.27 meters (5.0) feet and 2.54 meters (10.0 feet). The BESA was lowered into the bore hole immediately after the thin wall tube was removed. The soil zone tested by the BESA was the same as that sampled and subsequently tested by the triaxial test method.

Triaxial reload tests were performed on the two undisturbed samples. Modulii of elasticity, Er were plotted versus confining pressure. The BESA data was reduced. Secant and Tangent modulii (Es and Et) were developed. Results of all modulii obtained are shown in figure 33.

Some mechanical problems occurred with the BESA during the field testing at this site. Membrane leakage prevented the full range of membrane pressures from being applied. However, sufficient pressure was maintained to provide meaningful data, as will be shown.

In order to provide meaningful range of modulus versus confining pressure, an alternate method of extending the slope using 150 psi times E per psi of confining pressure was used. Settlement "predictions" were then made using the modulii developed by the BESA and layer strain analysis. The results are compared with actual measured settlements of the test mat and the hangar footings. Additionally, the results are compared with predictions made with triaxial reload modulii and layer strain analysis. Tables 7 and 8 are presented to provide data information resulting from the analyses.

Before discussing results, it is pointed out that the significance of test results is limited by the fact that only one test location (the boring adjacent to the test site) was used, and undisturbed samples as well as BESA tests, were limited to that one boring. Nevertheless, the soil profile (see figure 34) shows a relatively uniform consistency in depth, and this fact coupled with results obtained increase the confidence level that samples and BESA tests were performed in soils quite representative of those in the foundation area.

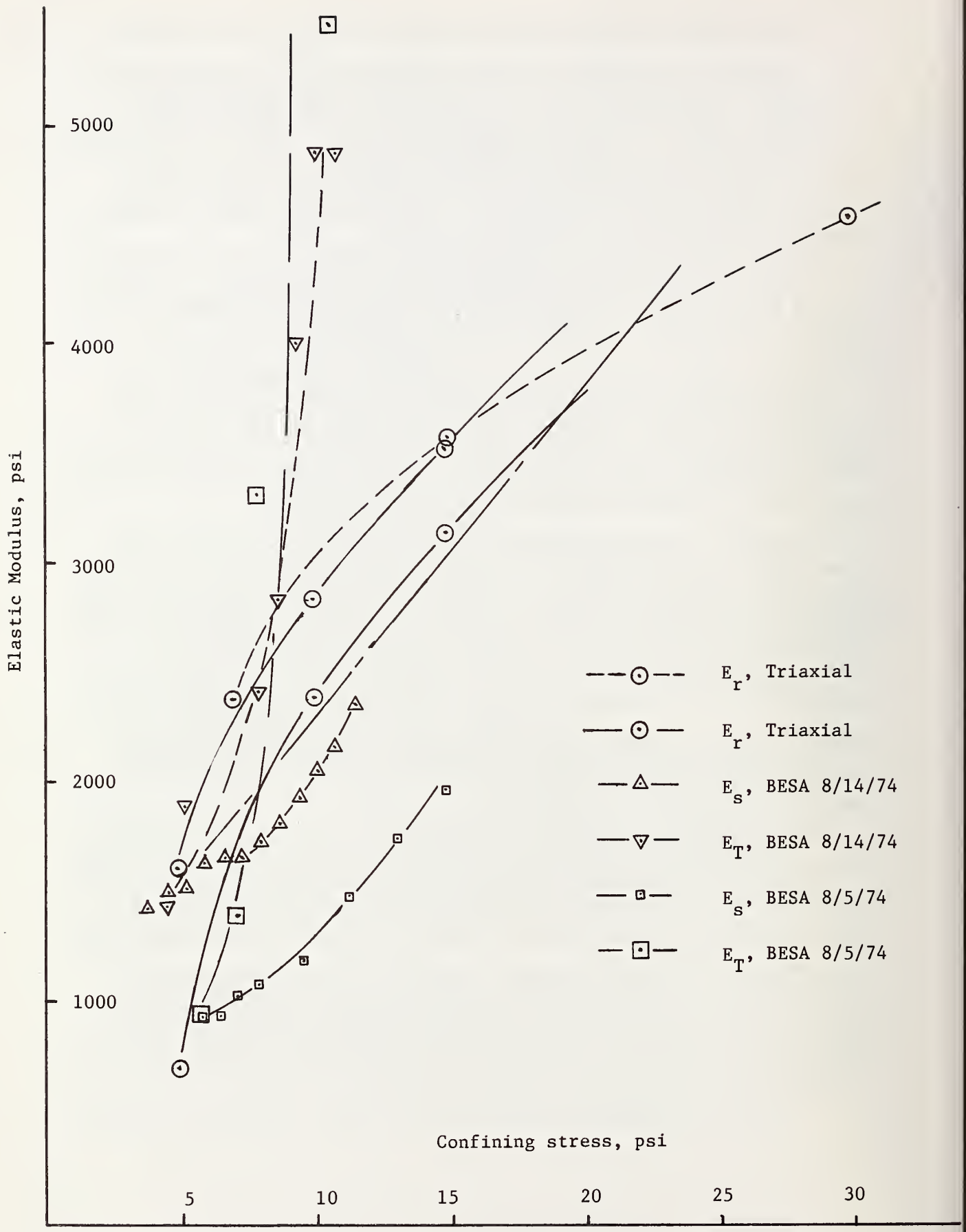


Figure 33 - Modulus Data, Donaldson Air Force Base

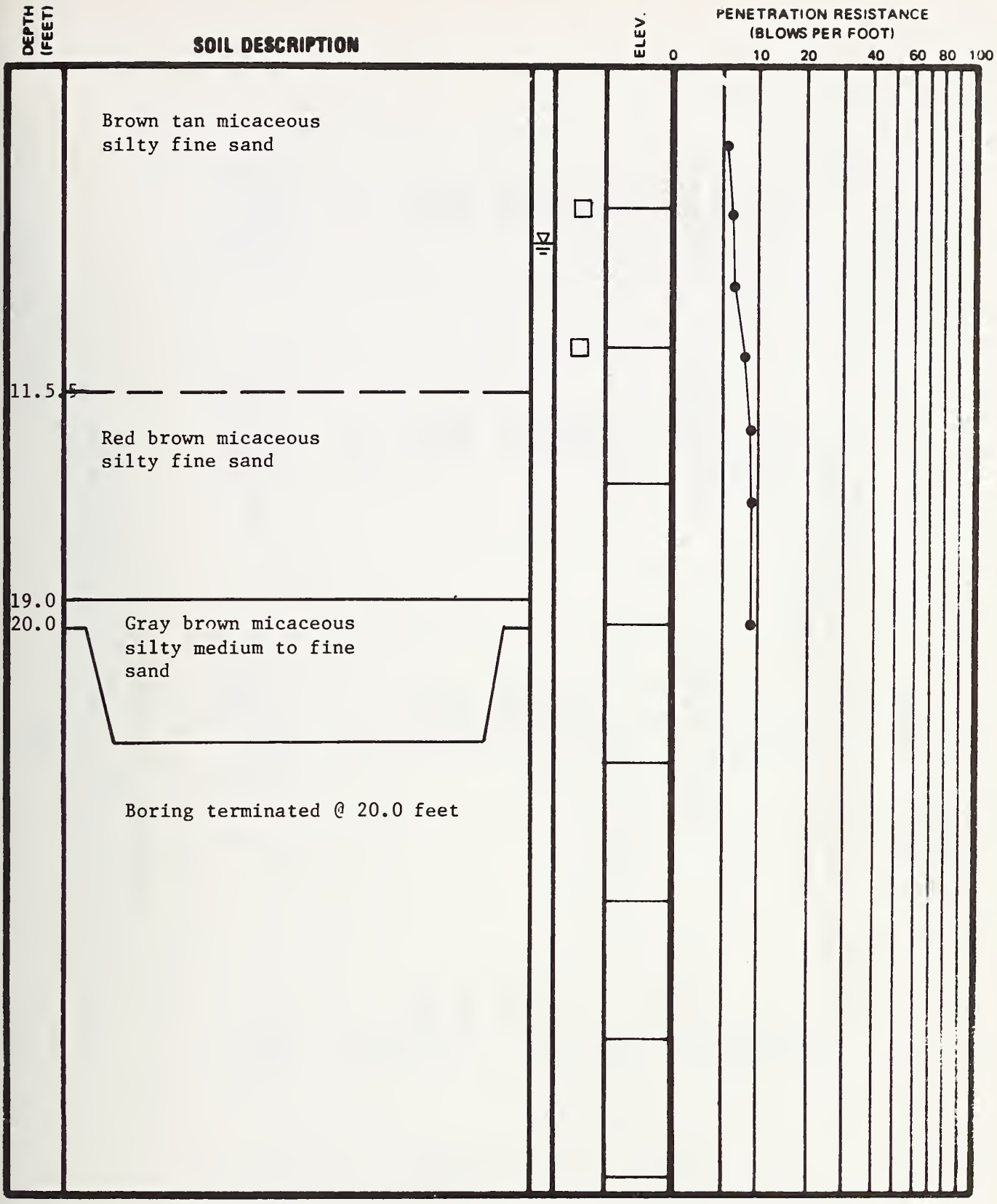


Figure 34 - Donaldson Air Force Base Soil Test Boring

TABLE 7

Settlement Calculations Comparing BESA Modulus Data,
Triaxial Modulus Data, and Measured Settlements
Donaldson Center Mat Load Test

Load Tons	Pressure PSF	Settlement Triaxial Modulus, E_r inches	Settlement BESA Modulus, E_s inches	Measured Settlement inches
50	309	.202	.203	.192
100	617	.355	.401	.255
150	926	.480	.648	.400
200	1235	.570	.699	.507
250	1543	.695	.822	.660

Note: To convert psf to N/m^2, multiply by 47.9.
To convert inches to centimeters, multiply by 2.54.

TABLE 8

Settlement Calculations Comparing BESA Modulus Data,
Triaxial Modulus Data, and Measured Settlements
Donaldson Center Hangar Footings

Footing	Pressure PSF	Settlement Triaxial, Modulus, E_r inches	Settlement BESA Modulus, E_s inches	Measured Settlement inches
E - 1	1310	0.51	0.605	.46
H - 1	1310	0.51	0.605	.40
H4	3040	0.96	1.124	.79
E4	3040	0.96	1.124	.43
ES	3010	0.95	1.116	.67
H5	3010	0.95	1.116	.94
H7	3040	0.96	1.124	.86
E7	3040	0.96	1.124	.78
E8	2360	0.80	0.943	.77
H8	2360	0.81	0.943	.78
H10	1460	0.55	0.660	.55
E10	1460	0.55	0.660	.58

Note: To convert psf to N/m^2, multiply by 47.9.

To convert inches to centimeters, multiply by 2.54.

In the mat load test, the measured settlement at maximum load of 250 tons was 1.68 centimeters (0.66 inches). The settlement predicted by BESA modulii and layer strain analysis was 2.08 centimeters (0.82 inches). That predicted by triaxial reload data and layer strain analysis was 1.78 centimeters (0.70 inches). The settlement predicted by consolidation testing by the Corps of Engineers, was 12.67 centimeters (4.99 inches).

In comparing these predictions the following "percent errors" result:

BESA modulus - overestimated by 24.2%
Triaxial modulus - overestimated by 6.0%
Consolidation - overestimated by 560.6%

The margin of error in predicting for both the BESA modulus method and the triaxial reload modulus method were acceptable and reasonable. The traditional consolidation approach proved highly erroneous, which is not unusual in Piedmont residual soil settlement analysis using consolidation test data.

In the hangar footing settlement analysis, actual measured settlements at maximum loading 69,934 N/m^2 (1460 pounds per square foot) were as shown in Table 3. Since the BESA modulus data and the triaxial modulus data were obtained from soils only at one boring location, it is reasonable to assume that some of the subsoils existing below tested footings were not of similar consistency and engineering characteristics. The settlements measured at footings E4 and E5 were significantly less than predicted by either the BESA method or the triaxial method. Therefore it is suggested that settlement predictions in ten of the twelve footing areas be further analyzed, and these two footing area measurements be arbitrarily eliminated due to apparent non-representation. For the remaining ten footings, the BESA method overestimated settlements an average of 0.43 centimeters (0.17 inches). The triaxial reload method overestimated the ten measured footing settlements by an average of 0.18 centimeters (0.07 inches). All twenty footing settlement predictions were overestimates. The average measured settlement was 1.75 centimeters (0.69 inches). The average percent error was as follows:

BESA modulus = 24.6%

Triaxial modulus = 10.1%

Consolidation = Not available

The degree of accuracy for settlement predictions with the triaxial modulus and the BESA modulus is considered reasonably good for Piedmont residual soils. Certainly in view of the limited testing it is surprisingly accurate.

3. Mat Foundation: The sixteen story Atlanta Internationale Hotel in Atlanta, Georgia is founded on a mat foundation. Most high-rise structures in the Atlanta area are supported by deep foundations such as caissons (drilled piers), or piles. Since the Atlanta Internationale Hotel did represent a rather unusual foundation system, actual settlements were carefully monitored during construction using fifteen control points and a level.(7)

The foundation in the sixteen story portion of the hotel structure consisted of a reinforced concrete mat 20.12 meters (66.0 feet) by 62.48 meters (20.5 feet) in plan dimensions and 1.14 meters (3.75 feet) thick. The mat was constructed at the bottom of a 3.66 meter (12 foot) deep excavation. The bearing soils within 18.3 meters (60 foot) depths were primarily very firm silty micaceous sands and stiff to very stiff sandy micaceous silts. Penetration resistances of these soils generally ranged from 15 to 25 blows per foot. Higher consistency soils and partically weathered rock occurred at an average depth of about 22.9 meters (75 feet) below the mat bearing elevation. Ground water was measured at a depth of approximately 15.2 meters (50 feet).

Conventional consolidation analyses were performed using undisturbed samples obtained in the soil test boring program. Settlement predictions ranged from 7.6 centimeters (3.0 inches) to 12.7 centimeters (5.0 inches), with an average of 10.2 centimeters (4.0 inches).

Settlement was recorded from initiation of the construction, until total settlements had occurred. At maximum load, calculated to be 1840 pounds per square foot (88,136 N/m^2). Settlement nearest the center of the mat measured to be 3.15 centimeters (1.24 inches).

A layer strain method was applied to predict settlements of the mat. Modulus data was obtained from two sources: in-situ BESA test data, and slow cyclic triaxial test data on undisturbed soil samples. Results of modulus testing are shown on figure 35. The soil stratum was divided into five layers as shown in figure 36 for subsequent analyses The stress changes due to the building load were computed using Westergaard influence factors at the mid points of each layer. It was assumed that the very dense soils below the "compressible" zones were incompressible, and that settlement contributions from this zone were negligible.

The lateral confining pressure at the center of each layer was calculated using Ko = 0.5.

Results of the settlement predictions were as follows:

Actual measured settlement:	3.15 centimeters (1.24 inches)
BESA modulus prediction:	3.61 centimeters (1.42 inches)
Triaxial modulus prediction:	3.48 centimeters (1.37 inches)
Consolidation prediction (average):	10.16 centimeters (4.0 inches)

Percent errors resulting are:

BESA modulus = 14.5%
Triaxial modulus = 10.5%
Consolidation = 222.6%

The accuracy of both the BESA modulus testing and the triaxial modulus testing is excellent, while the settlement prediction using consolidation data and theory, is highly inaccurate.

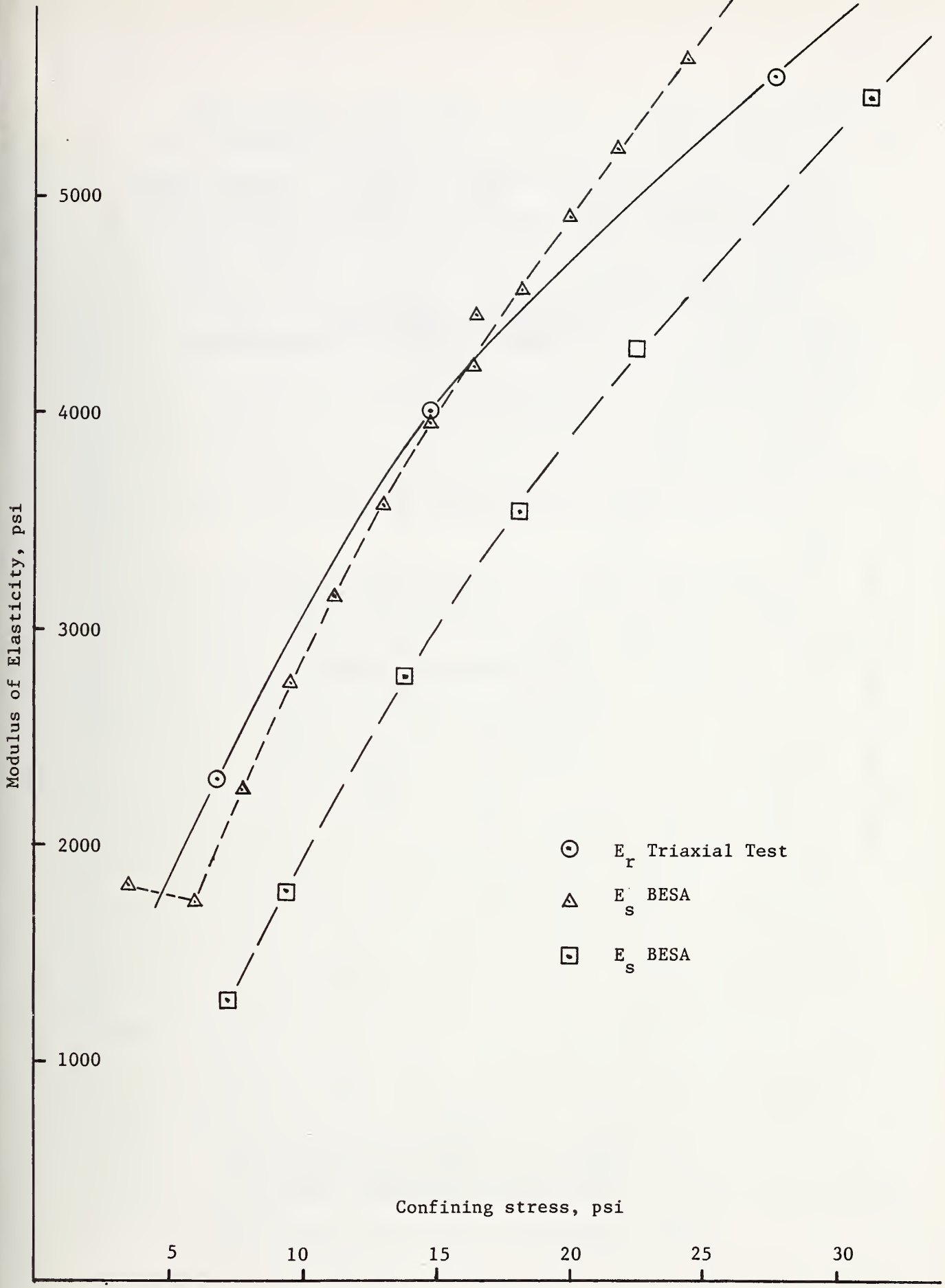


Figure 35 - Modulus Data - Atlanta Internationale Hotel

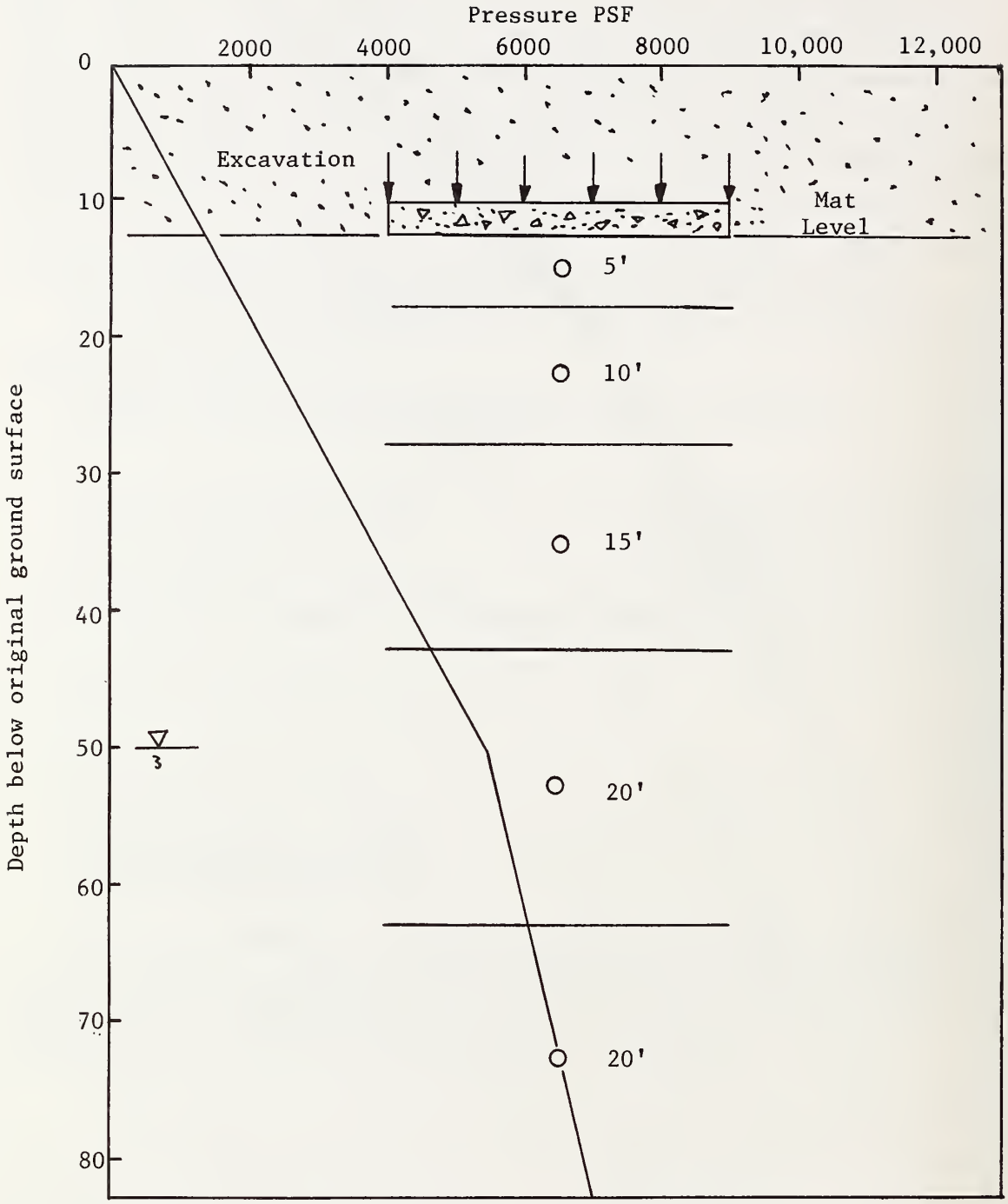


Figure 36
Layers used in Layer-Strain Method
and variation of σ_o

4. Similitude Model Load Tests in Sand: In 1965, twenty-two BESA tests were performed within bore holes in an aolian sand located in Story County, Iowa.[3] These tests represented eleven sets using two different sized model "plates", the larger being 1.5 times the linear dimension of the smaller. The larger set of plates measured 3.0 inches by 4.0 inches (7.62 cm x 10.16 cm), while the smaller set of plates measured 2.0 inches by 2.67 inches (5.07 cm x 6.78 cm). Six sets of tests were performed with membrane pressure applied. Five sets of tests were performed with no membrane pressure applied. One prototype plate load test, utilizing a 38.1 by 50.8 centimeter (15 by 20 inch) plate was performed in the same soil. The ratio of linear dimension or scale ratio n, of the prototype plate to the two sets of model plates was 5.0 for the larger (3" x 4") model plates and 7.5 for the smaller (2" x 2.67").

Bore holes were drilled in the soil by means of hand augers. The holes were then smoothed and slightly enlarged by means of a hand-operated reamer, or cutting tool. This reamer consisted of a cylindrical pipe with an outside diameter of 16.5 centimeters (6.5 inches). The lower edge of the pipe was sharpened with the bevel on the inside to minimize soil disturbance near the walls of the bore holes.

The BESA was then lowered into the bore hole to the desired test depths by means of a wire cable fastened to a quick-release clamp supported on a tripod. The apparatus was allowed to hang suspended prior to the application of the air pressure in the membrane (for the tests utilizing membrane pressure). The air pressure was then applied within the membrane, forcing the membrane against the sides of the bore hole, with a pressure equal to the sum of the desired surcharge pressure and a second component called the "sgp" or similated gravitational pressure. Hydraulic pressure was then applied to the larger set of model plates, forcing the plates against the rubber membrane. This plate load, transmitted through the membrane to the soil, caused the soil to compress. The plate load was maintained at a constant intensity until the rate of compression (settlement) was less than 0.002 inches or 0.005 centimeters per minute. Strain gage readings were then taken and recorded, and the next higher model plate load increment was applied.

After completion of the test, normally consisting of five or six load increments, the hydraulic pressure on the larger set of model plates was removed, and these plates were contracted. The membrane pressure was then increased to a greater pressure with the larger "sgp" corresponding to the smaller model plate set. Hydraulic pressure was then applied to the smaller model plate set, and load versus deformation (settlement) readings were again obtained. Care was taken to apply identical initial load pressures for the two plate sets. The additional load increments were also applied equally, so that a previously determined load restriction was the same for the "model" (in this case, the smaller plate set) and the "prototype" (the larger plate set).

Soil density measurements were made in field locations so that the unit weight of the soil being tested was known. This unit weight was necessary for calculations of sgp.

The sgp used in this testing was calculated to simulate the weight force of the soil at a constant depth of 2.54 centimeters (1 inch) for both model plate sets. The distortion induced by this method of sgp application was evaluated by analyzing the results of the tests performed.

The sand tested is a fine sand classified by the Soil Conservation Service, Story County Soil Survey Report as a "Thurman loamy fine sand". This sand was selected for testing because of its relatively uniform nature, its very low cohesion, and the large amount of information previously obtained from it. Soil identification tests performed on this soil were: moisture contents, liquid limits, plastic limits, grain size analyses. In addition to these identification tests, laboratory direct shear, laboratory triaxial shear, and field bore hole shear (BHS) tests were also made in this soil. Finally, density tests and unit weight determinations were made. Results of all tests are shown in table 9.

Results of BESA field tests in the sand soil are shown in figures 37 through 42. "L" designates large plate tests, while "S" refers to small plate tests. The amount of settlement in the graphs was obtained by

TABLE 9

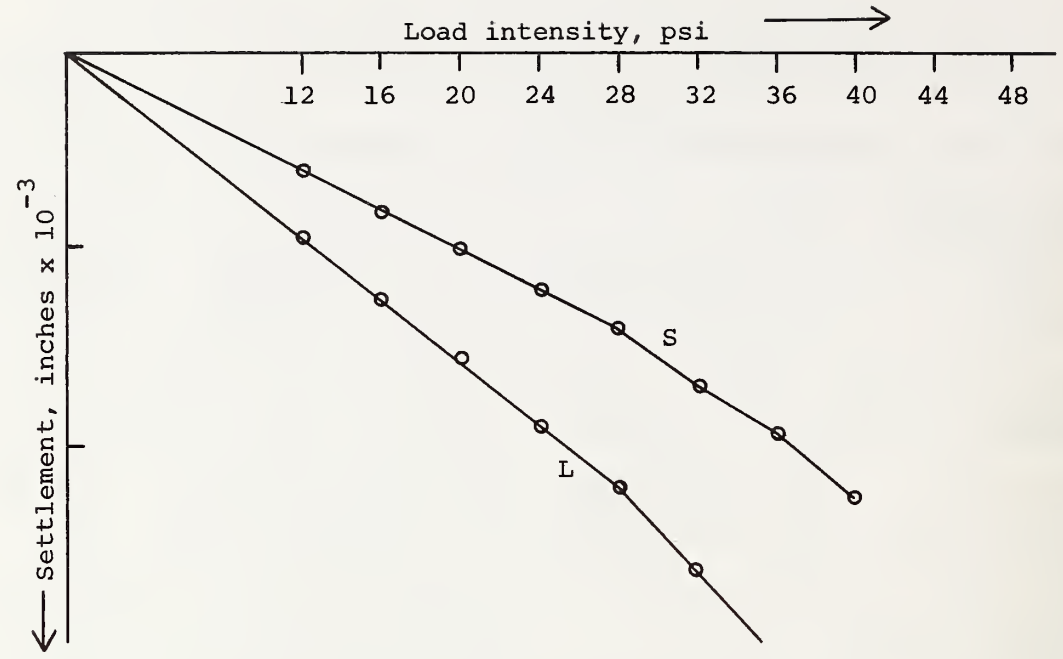
Soil Test Data

Soil Type	Dune Sand	Silt	Clay
Classification			
AASHO-ASTM	A-2-4 (0)	A-4 (8)	A-7-6 (20)
Unified	SM	ML	CH
Plasticity			
Liquid Limit, %	16.2	30	88.8
Plasticity Index	NP	5	58.7
Size Gradation, %			
Gravel (2mm)	0.0	0.0	0.0
Sand (2-.074mm)	87.9	1	0.6
Silt (.074-.002mm)	7.2	84	19.0
Clay (.002mm)	4.9	15	80.4
Dry Density, psf	99.0	83.3	93.2
Moisture Content, %	4-8	17	33

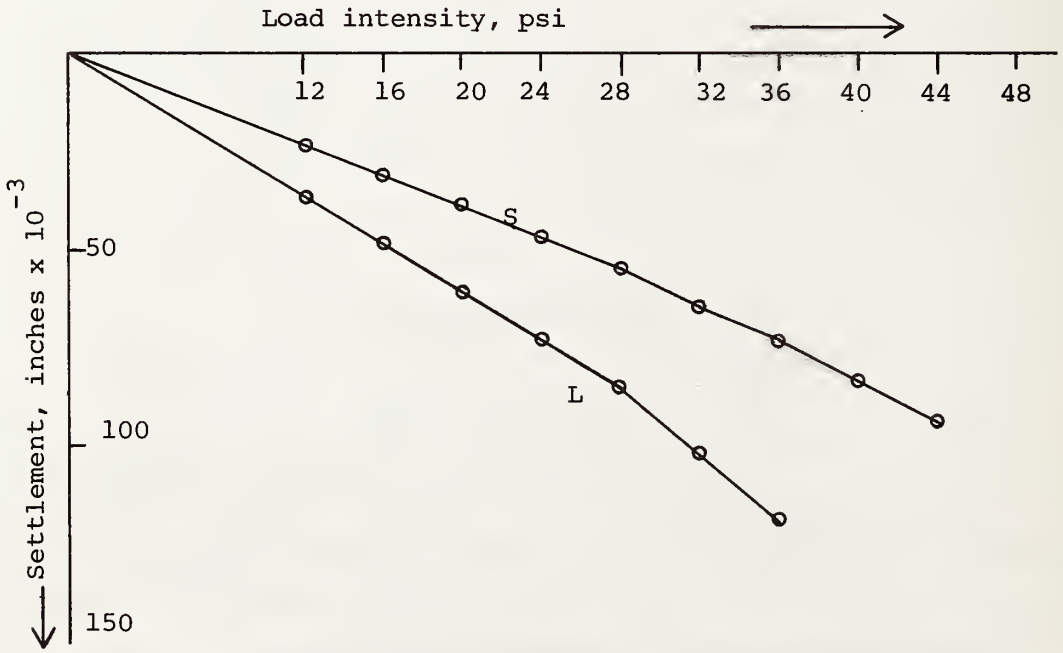
Shear Strength Parameters

	BHS	Direct Shear Ø	Triaxial	BHS	Direct Shear C	Triaxial
Sand	36.9°	36.5°	35.8°	0.4 psi	0.3 psi	0.3 psi
Silt	24.0-29.5°	24.1-24.7°	28.9°	0.7-4.3 psi	0.2-1.8 psi	2.4 psi
Clay	----	4.5°	----	----	15.0 psi	15.5 psi

Note: To convert psi to kN/m^2, multiply by 6.89.

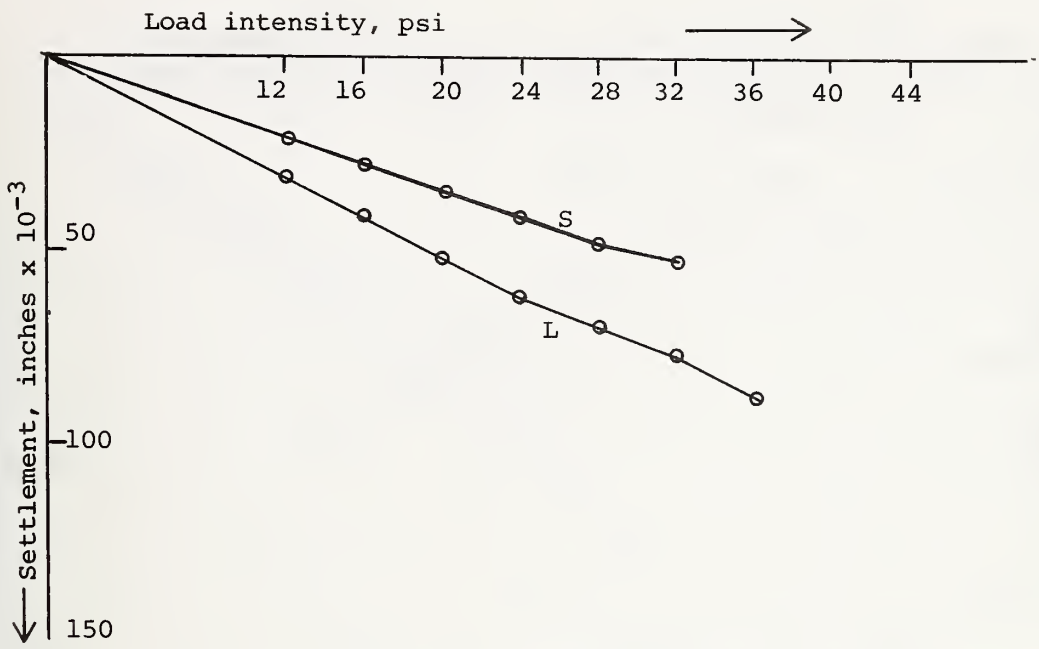


Field tests 1 and 2, sand

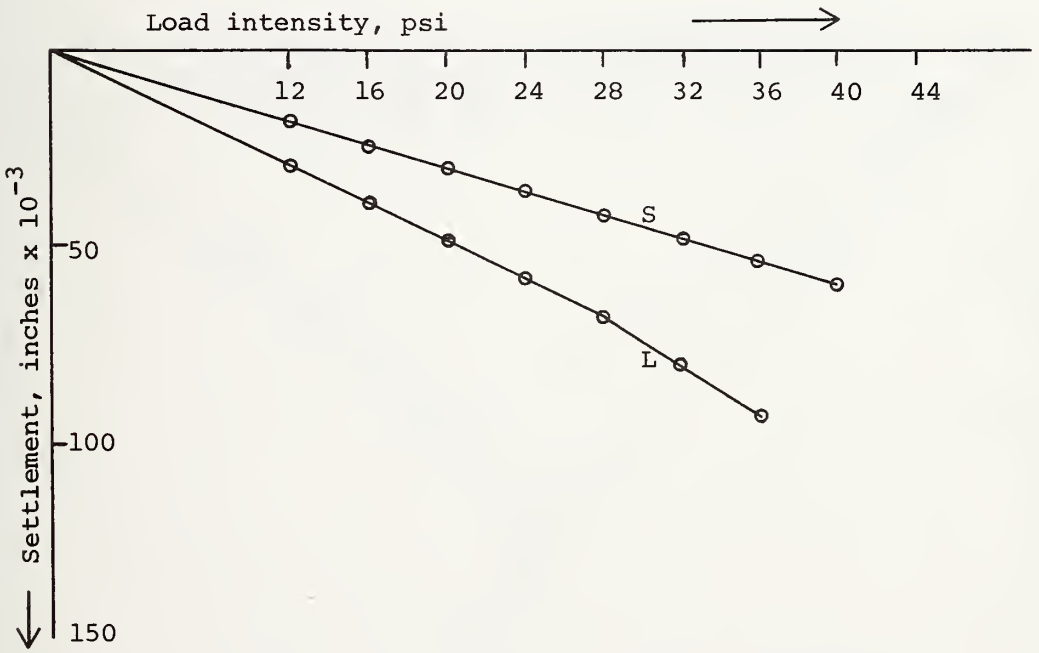


Field tests 3 and 4, sand

Figure 37 - BESA Similitude Tests, Sand

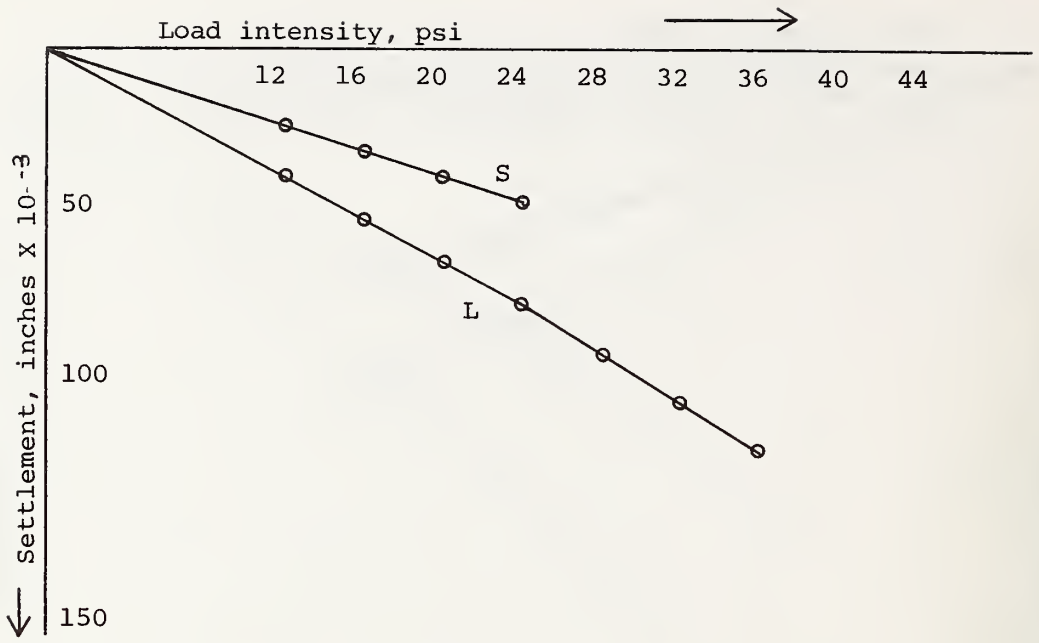


Field tests 5 and 6, sand

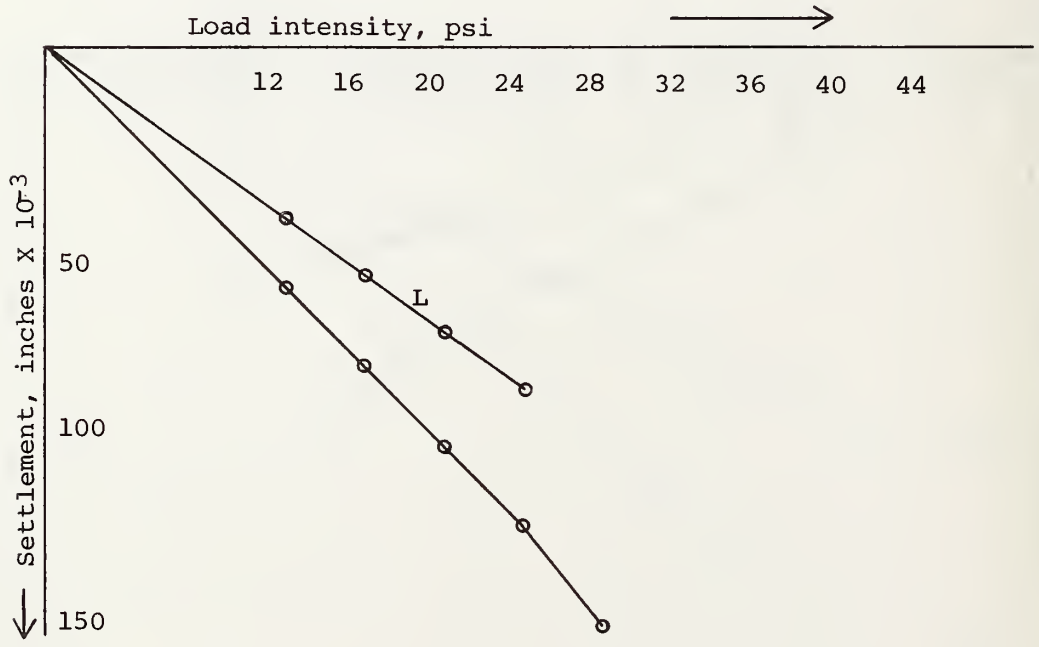


Field tests 7 and 8, sand

Figure 38 - BESA Similitude Tests, Sand

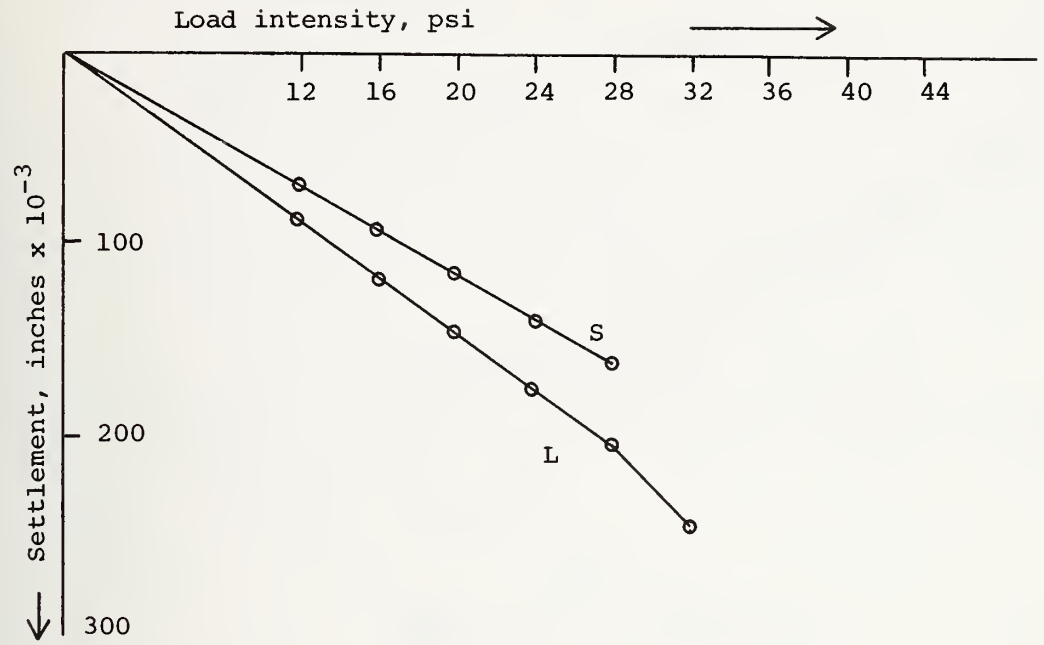


Field test 9 and 10, sand

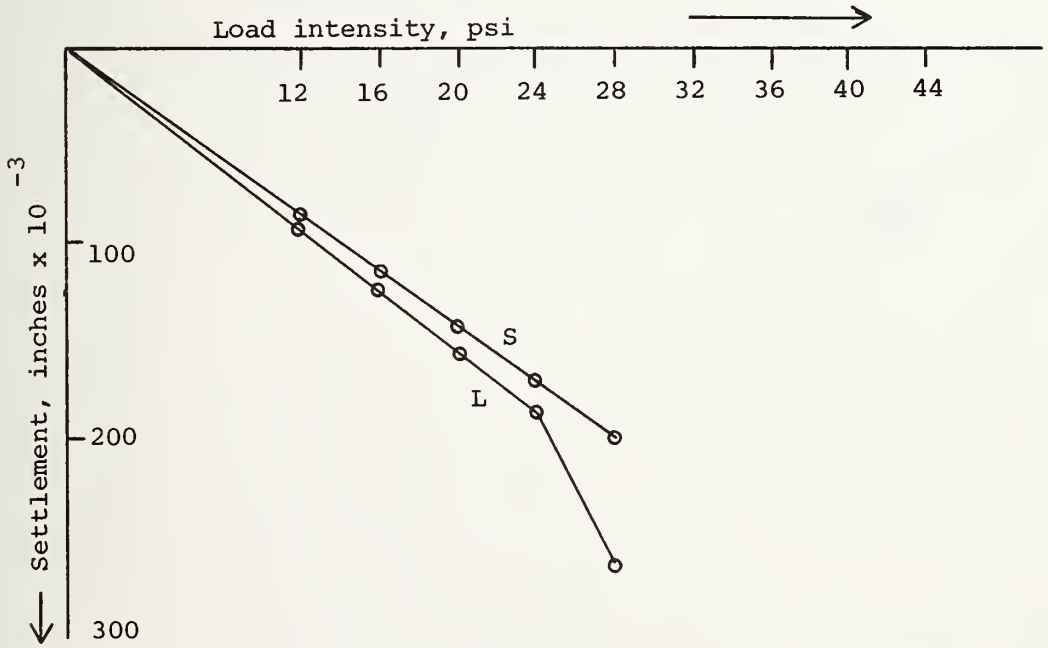


Field tests 11 and 12, sand

Figure 39 - BESA Similitude Tests, Sand

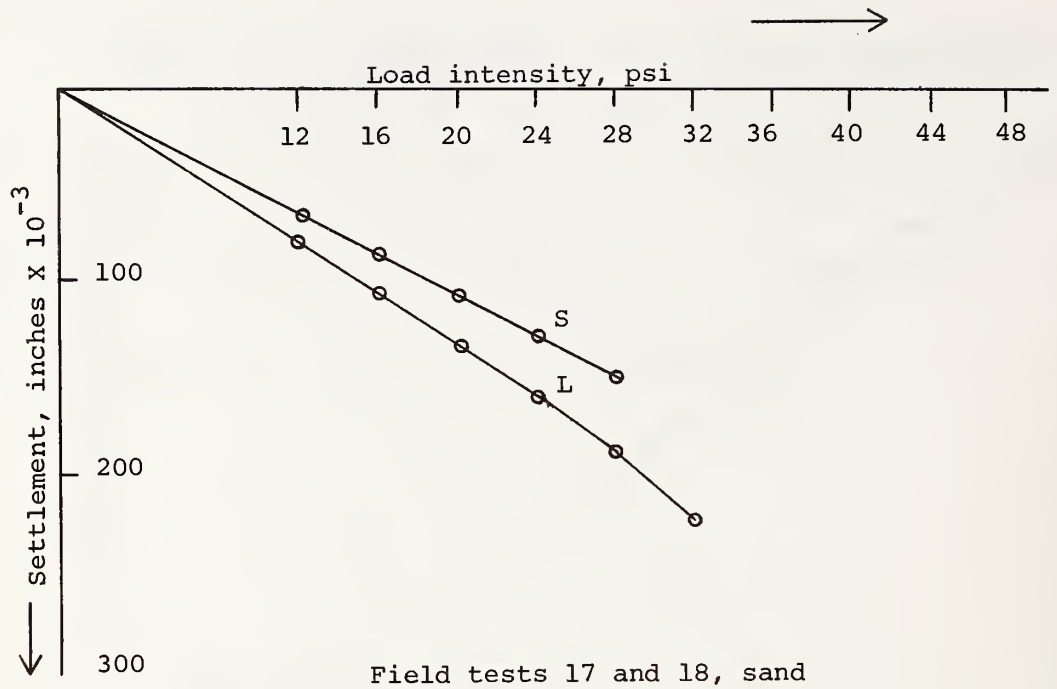


Field tests 13 and 14, sand

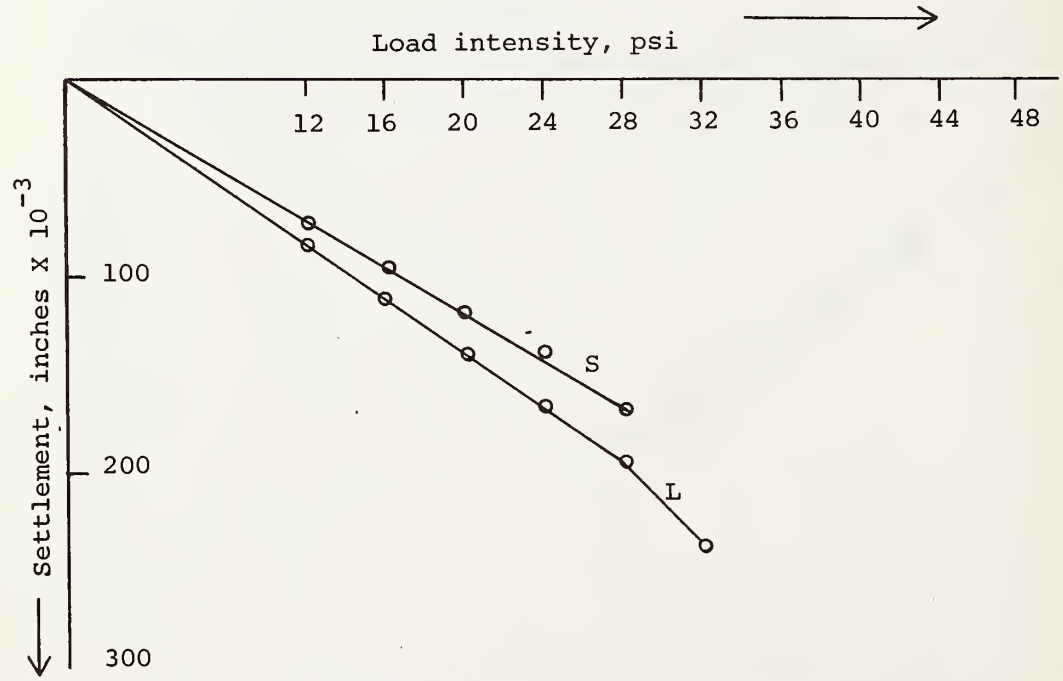


Field tests 15 and 16, sand

Figure 40 - BESA Similitude Tests, Sand



Field tests 17 and 18, sand



Field tests 19 and 20, sand

Figure 41 - BESA Similitude Tests, Sand

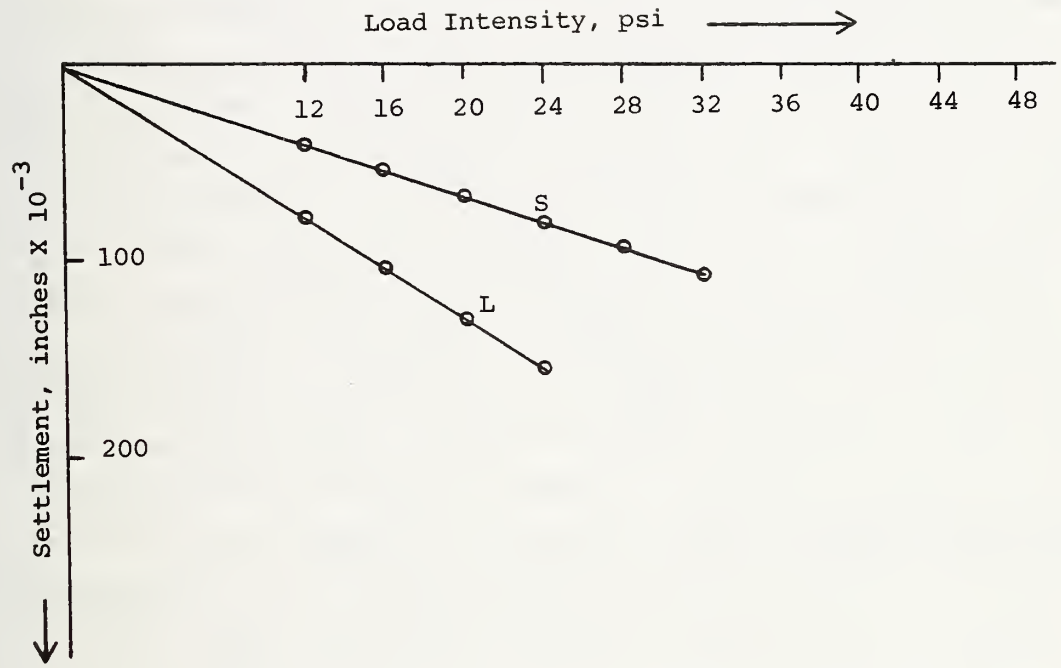


Figure 42 - BESA Similitude Tests, Sand

dividing the amount of settlement of both opposing plates by 2. The prototype settlement test with a 38.1 by 50.8 centimeter (15 by 20 inch) plate is shown in figure 43 .

Time-settlement information was not obtained. The time of duration of load application in these tests was determined by the rate of settlement and not by an arbitrary or predetermined duration.

The BESA tests in sand show a very fine straight line load versus settlement curve until at least 192.9 kN/m^2 (28 psi) for all tests performed, with and without sgp membrane pressure. The curves generally began to show a decrease in slope between 192.9 kN/m^2 and 220.5 kN/m^2 (28 and 32 psi). The quantitative values of settlement corresponding to a load of 137.8 kN/m^2 (20 psi) are shown for all tests performed in sand in table 10. Also shown on this table is a ratio of settlement of the larger model plate divided by the settlement of the smaller model plate, again corresponding to a load intensity of 137.8 kN/m^2 (20 psi). Tests 1 through 12 were conducted with membrane pressure. The membrane pressure employed was the sum of three component air pressures. The first component was the constant opening air pressure required to inflate the membrane against the sides of the hole. The second component was the surcharge pressure corresponding to γ_Z ; , where Z is the depth from the surface to the place of the center of the prototype footing. The third component of air pressure is the simulated gravitational pressure, sgp. The amount of sgp varies with model plate size, and was determined by the equation γ sgp = $\frac{\gamma_n}{1728}$ where sgp is in psi and γ is in pcf.

The prototype was a 38.1 by 50.8 centimeter (15 by 20 inch) metal plate or footing, therefore n = 7.5 for the small model plates and n = 5.0 for the larger model plates in the BESA.

A rejection criteria was established for the data obtained. Any measurement was rejected when the magnitude of its deviation from the mean was such that the probability of the occurrence of all deviations that large or larger was less than 1/2 m, where m is the number of measurements taken. Two possible causes for incorrect or non-representative readings were instrument error and soil irregularities.

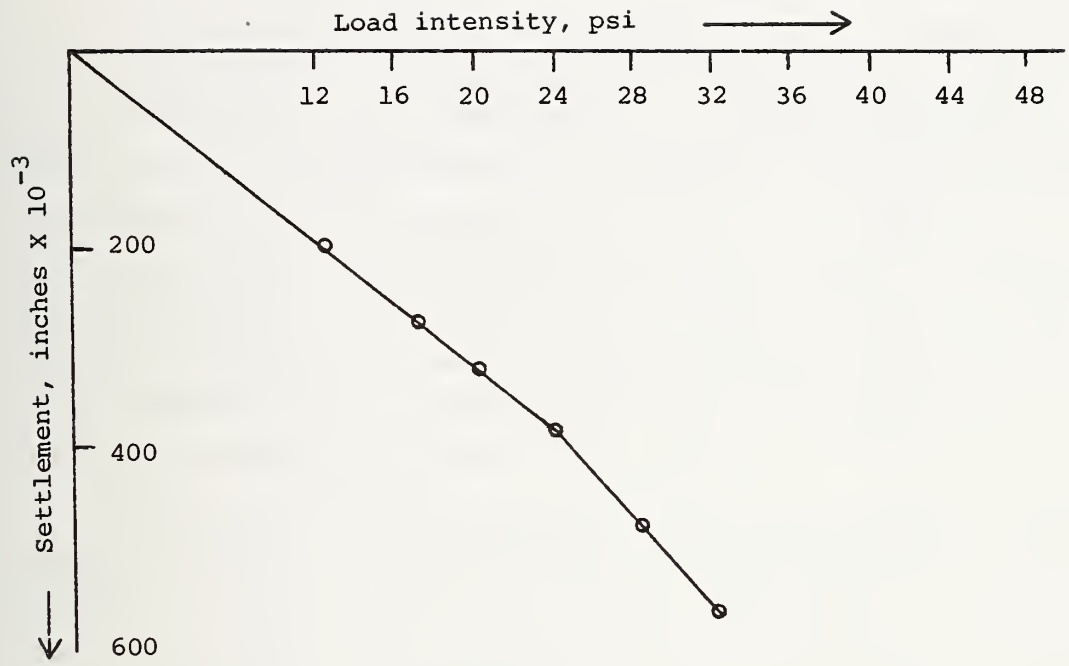


Figure 43 - 15" by 20" Prototype Plate Load Test,Sand

TABLE 10

BESA Similtude Data
Sand Site

Test	Settlement Ratio	3 Inch Plate Settlement	2 Inch Plate Settlement
1 & 2	1.56	.0759 inch*	.0507 inch*
3 & 4	1.55	.0600 inch	.0387 inch
5 & 6	1.47	.0512 inch	.0350 inch
7 & 8	1.41	.0495 inch	.0350 inch
9 & 10	1.62	.0550 inch	.0340 inch
11 & 12	0.71*	.0730 inch*	.1020 inch*
Mean Value	1.53	.0539 inch	.0357 inch
Standard Deviation	0.083	.0047 inch	.0011 inch
13 & 14	1.28	.144 inch	.112 inch
15 & 16	1.10	.156 inch	.142 inch
17 & 18	1.21	.133 inch	.109 inch
19 & 20	1.17	.140 inch	.120 inch
21 & 22	1.89*	.130 inch	.069 inch*
Mean Value	1.19	.1406 inch	.1208 inch
Standard Deviation	0.076	.0102 inch	.0148 inch

*Rejected data.

Note: To convert inches to centimeters, multiply by 2.54.

5. Similitude Model Load Tests In Clay: Case history number 4 discussed BESA testing employing similitude theory and "sgp" pressure in sands. Results were quite accurate, and prediction errors were low, on the order of 14% when data were used to predict prototype footing settlement. Similarly, 18 BESA tests were performed in a low cohesion silt with comparatively similar results.[3] A mean value of 1.56 for a settlement ratio of larger model plate to smaller model plate was achieved. This was close to the theoretical ratio of 1.50. The error in predicting prototype footing settlement was even lower than in sand, 11.7%. Once again, tests were performed without membrane pressure, and without sgp applied. The mean value of the ratio of large model plate to small model plate when BESA tests were performed without membrane pressure was only 0.65 rather than 1.50. Table 10 describes test data obtained from the silt soils evaluated.

The prototype footing in silt settled 0.325 centimeters (0.128 inches). The mean predicted settlement using membrane pressures and similitude theory was 0.361 centimeters (0.142 inches). The predicted settlement without membrane pressure was 0.239 centimeters (0.094 inches) for the 3 inch model plate and 0.544 (0.214 inches) for the 2 inch plate. This corresponds to predicted settlement of 0.368 centimeters (0.145 inches) for the 3 inch plate and 0.356 centimeters (0.140 inches) for the 2 inch plate with membrane pressure applied. Again, as with the sand tests, it was found in a silt that settlement prediction of a prototype footing was quite accurate using the BESA test data obtained with membrane pressure applied and the similitude approach. Again, as in sands, it was found that predictions of the prototype footing without the BESA membrane pressure resulted in highly erroneous predictions.

This last case history, with the BESA used in cohesive clay, represents a different situation in which settlement can be expected to occur by consolidation, and by water dissipating from soil pores rather than by compression of gas filled voids upon load. The clay soil tested was an alluvial clay located within the Missouri River floodplain. Field density measurements were made and unconfined compressive strength tests and direct shear tests were performed on remolded samples obtained from the same area. Results of these and soil identification tests as shown in table 11.

The BESA tests in clay resulted in a straight line plot to a load intensity of at least 82.7 kN/m^2 (12 psi) for all tests performed. The clay BESA tests were performed with the improved "second generation" BESA, while previously performed tests in sands and silts were performed with the initial more crudely constructed BESA model. Eight tests were performed with membrane pressure applied. Two tests were performed with no membrane pressure. Figures 44 through 46 show results of these tests. Table 10 lists the settlement ratios for each set of tests performed in the same hole the values of settlements corresponding to a 12 psi load intensity, and statistical information. This table also shows the results of two tests performed with no membrane pressure.

All four sets of tests with membrane pressure resulted in settlement ratios of large model plate settlement divided by small model plate settlement in the vicinity of 1.50, with the value of 1.65 being furthest from 1.50. A mean value of 1.57 resulted. The settlement ratio of the one set of tests in clay without membrane pressure was 1.22.

Settlement predictions from model tests for the settlement of the 38.1 by 50.8 centimeter (15 by 20 inch) prototype footing in clay were excessive. The mean value of the larger model plate test prediction corresponding to a load intensity of 82.7 kN/m^2 (12 psi) was 0.859 centimeters (0.338 inches). The mean value of the smaller model plate test prediction at a similar load intensity was 0.823 centimeters (0.324 inches.) The mean overall prediction of prototype settlement was 0.841 centimeters (0.331 inches).

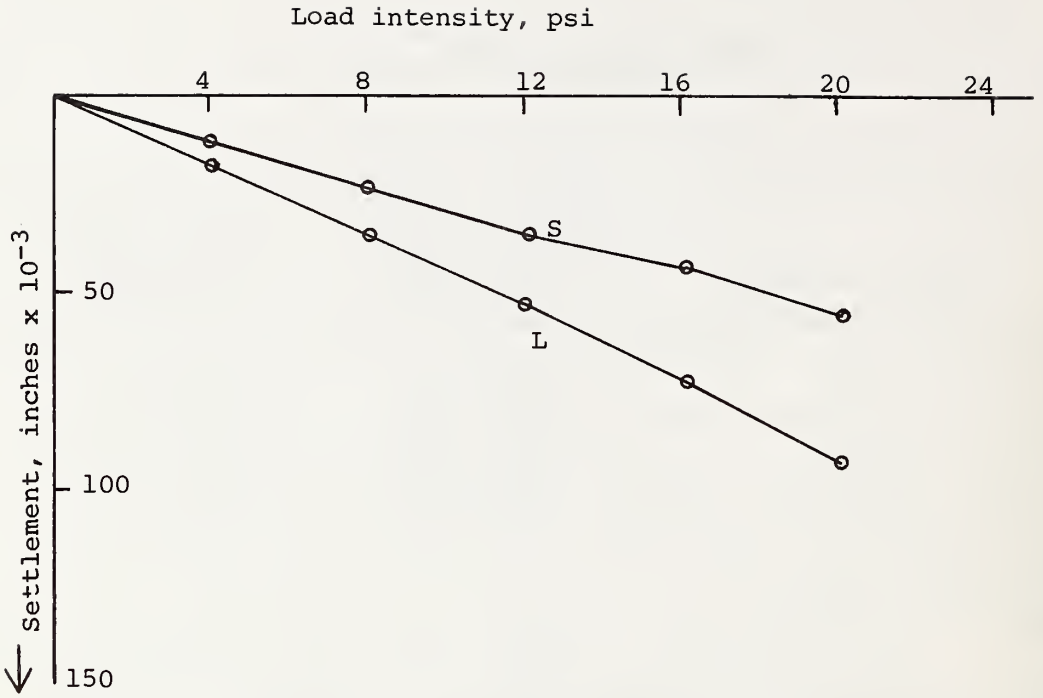
Actual prototype settlement was 0.328 centimeters (0.129 inches) corresponding to a load intensity of 82.7 kN/m^2 (12 psi). Thus the magnitude of error averages 157%. The prototype settled considerably less in clay than predicted. An ánalysis of test data, including evaluation of the time factor resulted in a conclusion that the degree of consolidation which was reached in the prototype test was less than that reached in the model tests. The reason that this was not observed in the sand and silt BESA tests was probably because the percent of settlement due to the consolidation phenomenon was almost non-existent in the sand and silt tested. The settlement in the sand and silt soils was almost entirely "immediate settlement" or elastic compression.

TABLE 11

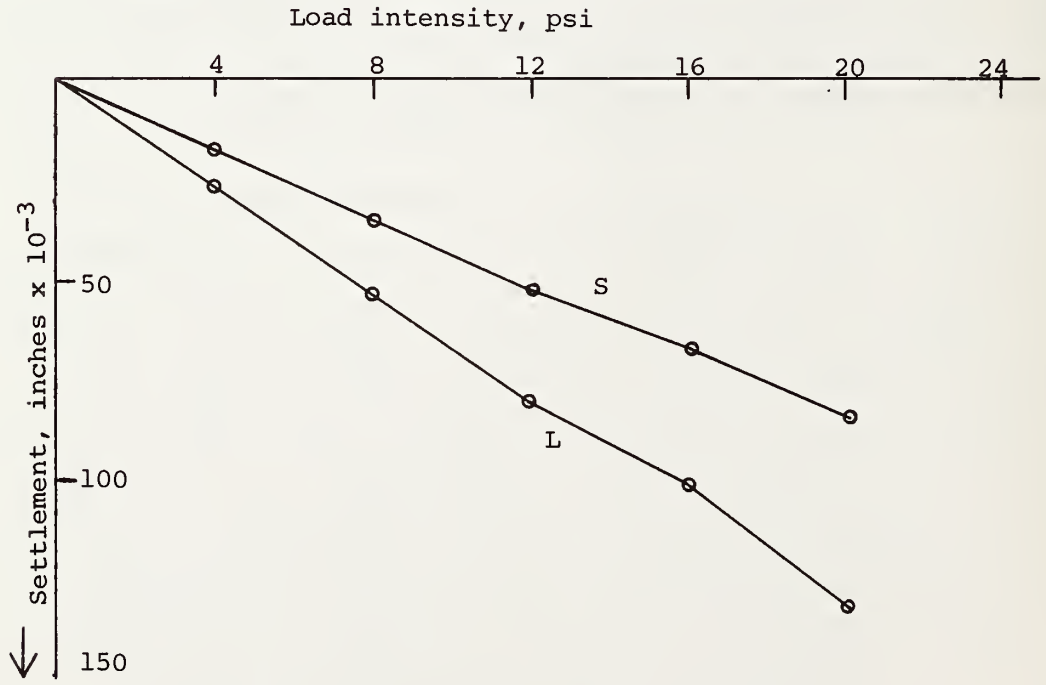
BESA Similitude Data
Clay Site

Test	Settlement Ratio	3 Inch Plate Settlement	2 Inch Plate Settlement
41 & 42	1.51	.0545 inch	.0360 inch
43 & 44	1.51	.0800 inch	.0530 inch
45 & 46	1.64	.0625 inch	.0380 inch
47 & 48	1.61	.0740 inch	.0460 inch
Mean Value	1.57	.0677 inch	.0432 inch
Standard Deviation	.068	.0114 inch	.0078 inch
49 & 50	1.22	.0540 inch	.0440 inch

Note: To convert inches to centimeters, multiply by 2.54.

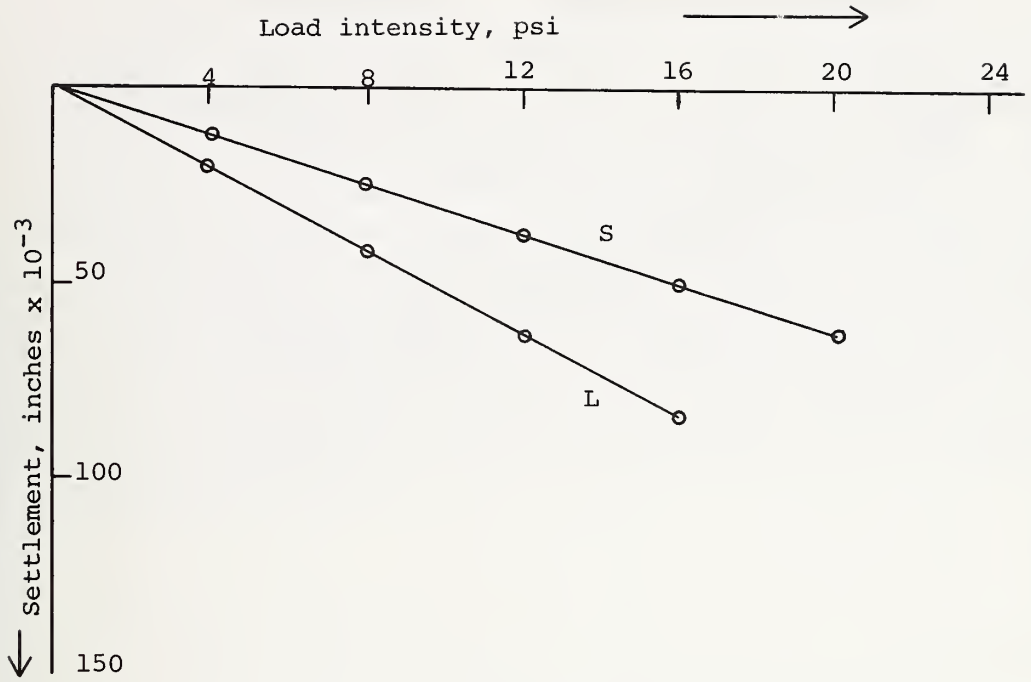


Field tests 41 and 42, clay

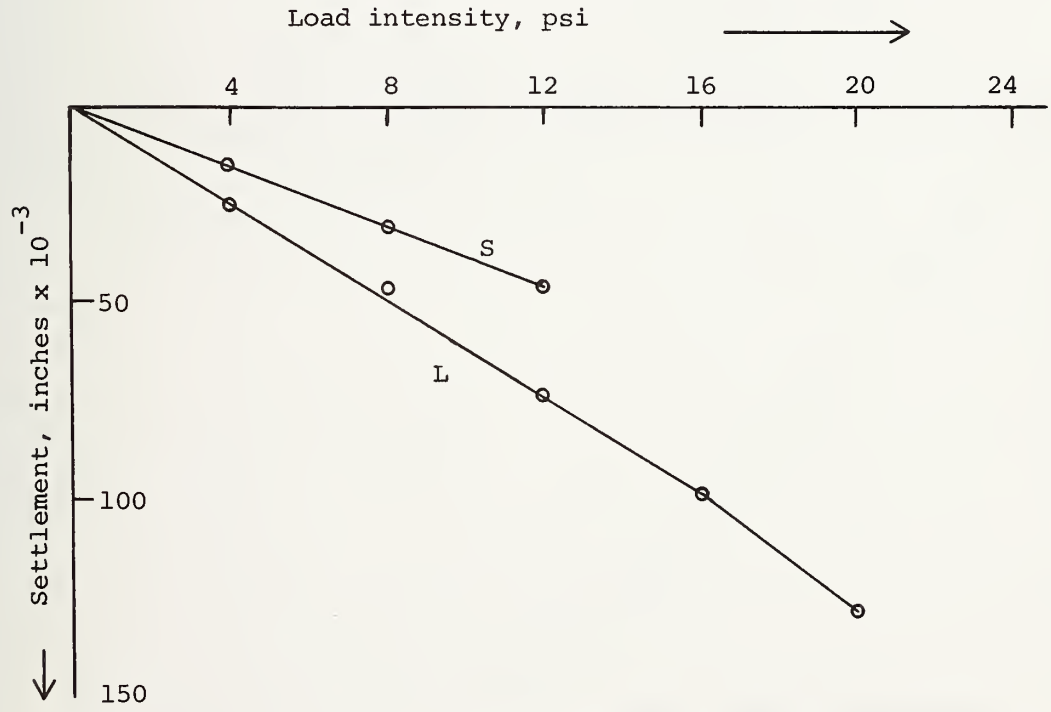


Field tests 43 and 44, clay

Figure 44 - BESA Similitude Tests, Clay

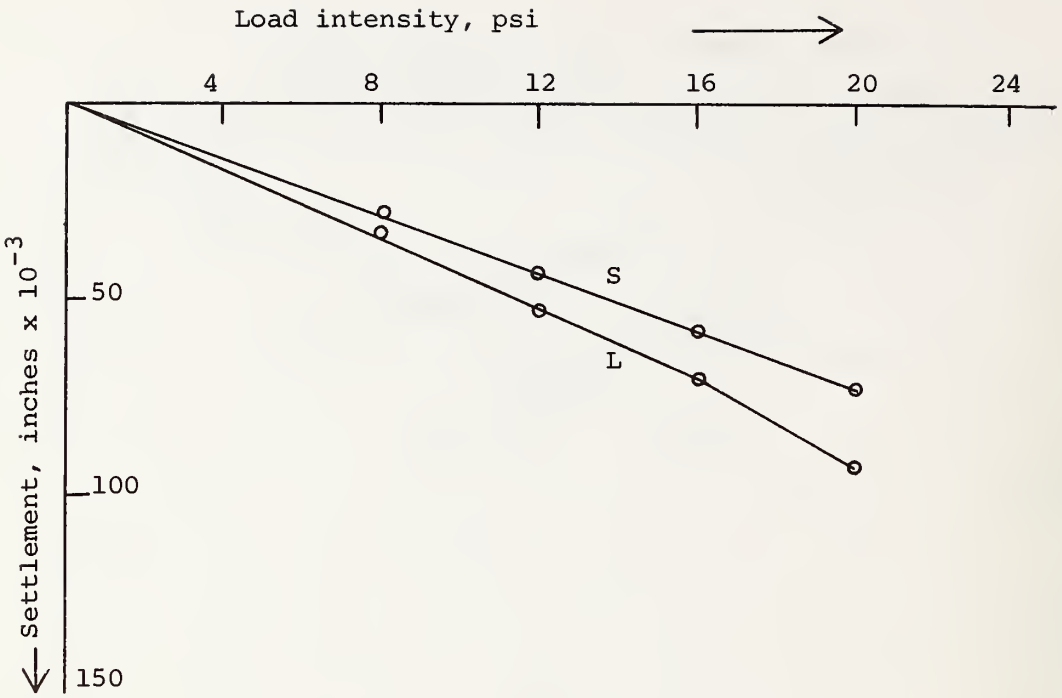


Field tests 45 and 46, clay

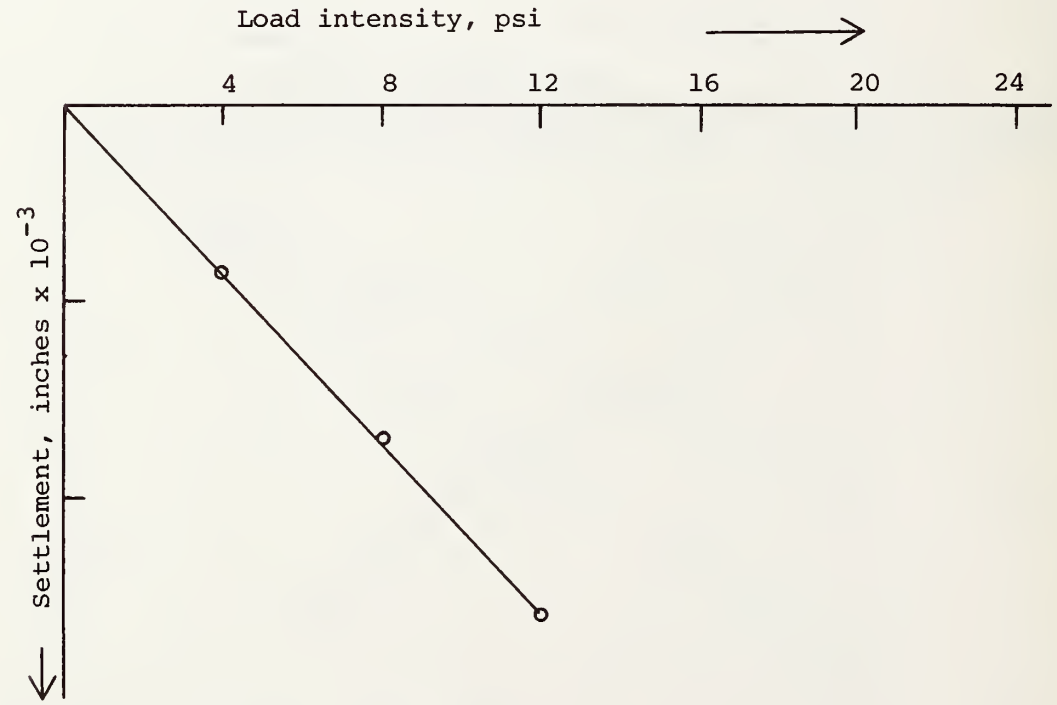


Field tests 47 and 48, clay

Figure 45 - BESA Similitude Tests, Clay



Field tests 49 and 50, clay



Prototype test, clay

Figure 46 - BESA Similitude Tests, Clay

The BESA test method used in a cohesive clay using similitude techniques appears to have serious limitations. It is recommended at this time, that BESA tests employing similitude techniques be limited to sands and granular silts, where little consolidation will occur, and settlement will be at least 80% "immediate" settlement rather than consolidation settlement. BESA testing in clay should utilize modulus theory and possibly an in-situ consolidation approach. As is evidenced in this case history, actual usage of the BESA has been so limited, that results when used in consulting situations, should be compared with other test data and methods until more confidence and more standardized techniques are developed.

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(2) McDaniel, G.T., "Bore Hole Earth Settlement Apparatus Application in Settlement Predictions in Piedmont Residual Soils", special problem submitted to the School of Civil Engineering, Georgia Institute of Technology in partial fulfillment for the Degree of Master of Science in Civil Engineering, 1974.

(3) Fox, Nathaniel S., "Field Model Tests for the Prediction of Foundation Settlement", a dissertation submitted to the Graduate Faculty in partial fulfillment of the Requirements for the Degree of Doctor of Philosophy, Iowa State University, 1966.

(4) Schwartz, Stanley, "Comparison of Horizontal and Vertical Consolidation", special problem submitted to the School of Civil Engineering, Georgia Institute of Technology in partial fulfillment for the Degree of Master of Science in Civil Engineering, 1971.

(5) Sowers and Sowers, "Introduction to Soil Mechanics and Foundations", 3rd Edition.

(6) Unpublished report, U.S. Army Corps of Engineers, Donaldson Air Force Base, 1952.

(7) Cugowski and Smith, "Settlement Prediction of a Mat Foundation on Residual Soils", special problem submitted to the School of Civil Engineering, Georgia Institute of Technology in partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering, 1972.

BORE HOLE ELECTRONIC EARTH PENETROMETER

(BEEP)

Introduction

The Bore Hole Electronic Earth Penetrometer (BEEP) is an instrument that could fill an immediate practical need. It generates soil consistency data by performing horizontal static penetration tests on the sides of a bore hole. The practical aspect of reacting or "jacking" against the sides of the hole rather than requiring a heavy dead weight reaction is a key factor in the future potential of this test method. Thus, small drilling equipment, or even hand drilling operations, can be used in conjunction with the BEEP.

Unfortunately, the BEEP is not yet in commercial production. Twenty five units were in the process of being manufactured, but a snag in the manufacturing process prevented their completion. Only one or two of these units have actually been completed. A modification in design is required prior to commercial manufacture.

Synopsis

The BEEP study discusses the correlation research which was conducted to compare BEEP data with both Standard Penetration Test (SPT) data and with dynamic hand operated penetration test data ("A & P"). Comparative results were excellent, with a standard error of approximately one "blow per foot." Subsequently the BEEP was used almost exclusively for a six-month period in a consulting capacity, and proved to be very quick and reliable. Monetary advantage include speed of testing as well as reduced labor requirements in comparison with the SPT. Uses in engineering design are discussed. Basically, any problems solved with standard penetration test data can be solved with BEEP data. Since most foundation investigations in the United States involve the standard penetration test, the potential usage of the BEEP is readily apparent. Finally, it is shown that the BEEP in conjunction with more portable drilling means, can be used in remote areas, where it would be impossible or impractical to use a large drill rig.

USES IN ENGINEERING DESIGN

The Bore Hole Electronic Earth Penetrometer (BEEP)[1] has been applied to many of the same general foundation design type problems in which the Standard Penetration Test (SPT) has been used. However, the non-availability of the BEEP for commercial purchase as of this date has severely limited its use in providing data for the solution to geotechnical problems. Three general types of geotechnical problems solved with the aid of the Bore Hole Electronic Earth Penetrometer data are: general subsurface investigations and feasibility studies; foundation design studies; and foundation/footing inspections. The application of the BEEP to these problems is discussed below.

Subsurface Investigation and Feasibility Studies: The Bore Hole Electronic Earth Penetrometer (BEEP) was used extensively by the author during a period of six months in place of Standard Penetration Tests and also in conjunction with Standard Penetration Testing to obtain field data relating to soil consistencies for subsurface investigations, engineering evaluations, and feasibility studies. These studies generally included an evaluation of site and subsurface conditions prior to the design of a structure or other facility in order to anticipate potential geotechnical problems and to provide soil consistency information upon which to select foundation systems. Furthermore, the data obtained was normally sufficient to provide a basis for foundation design.

As in the Standard Penetration Test, soil consistency information provided by the BEEP can be classified into three categories: soils which are adequate to support the proposed construction on shallow foundation systems; soils which are unsuitable to support proposed shallow foundation systems; and marginal soil conditions which might or might not be suitable and which would require further evaluation, generally by obtaining undisturbed samples, performing selected laboratory tests, and evaluating test results by utilizing soil mechanics theories and engineering judgements.

In addition to the consistency information which was correlated to "equivalent standard penetration test blow counts", disturbed auger cuttings were obtained and classified, and ground water information was measured and recorded. As implied, when marginal (and at times unsuitable) soils were encountered, undisturbed thin wall tube samples were obtained for laboratory testing evaluation. The advantages found in utilizing the BEEP for feasibility studies and subsurface investigations were: greater speed in data gathering; the lower costs of obtaining data; and portability of equipment into areas of limited access.

In shallow subsurface investigations for smaller sized projects and/or inaccessible sites where mechanical drill rigs could not traverse, the BEEP was used in conjunction with hand augering to provide soil consistency data. This solution to a geotechnical problem proved to be practicable in areas where the holes remained open and information was not required beyond a depth of 3 meters (10 feet), or at the most, 4.6 meters (15 feet). This method of avoiding the use of mechanical drilling equipment highlighted the potential advantage of flexibility and mobility. On other occasions, small mechanical drilling equipment such as the Simco SP900, which is self-propelled and can be lifted onto the back of a pick-up truck by two men, was used to mechanically advance a bore hole without the mobility limitations of a heavy drill rig. This particular drill rig moves on the ground with its operator walking behind it. It can traverse very soft terrain and heavily wooded areas. The drill rig, being too small to provide a Standard Penetration Test capability, is highly compatible with the portable aspects of the Bore Hole Electronic Earth Penetrometer test method.

The BEEP was also used with conventional drill rig equipment which had the capability of providing standard penetration testing as well as augering holes for BEEP testing.

Foundation Design: The use of the BEEP in foundation design is related to general subsurface investigations and feasibility studies, but it addresses the specific geotechnical application of providing data on which to design foundations. The second generation instrument which

was used exclusively during six months of major BEEP use, had an internal limitation on the maximum penetration force that could be applied. The maximum penetration pressure was limited to an equivalent standard penetration resistance of 22 blows per foot. Thus it could evaluate soils in the zero to 22 blow per foot range quantitatively and inform the investigator that soils were above 22 blows per foot when the capacity of the instrument was exceeded. In most investigations performed, this range of testing capability was adequate for shallow foundation design. Consistency data in terms of penetration test blow counts, and shallow foundation design was accomplished in a manner similar to that commonly utilized in the geotechnical consulting field for determining allowable foundation pressures. Furthermore, as in the Standard Penetration Test, very low consistency soils which were unsuitable or marginal would be further investigated through the use of undisturbed sampling and laboratory testing as previously mentioned. Soils with consistencies above 22 blows per foot could not be quantitatively evaluated with the second generation BEEP model with respect to allowable bearing pressures above that corresponding to 22 blows per foot soil.

It is acknowledged that the Standard Penetration Test has certain theoretical limitations, generally lacks sophistication, and is not uniformly accepted in all geographical regions or for all soil types. However, within the United States, the single greatest source of soil information used today in design is generally accepted to be the Standard Penetration Test. The BEEP was used in a similar capacity as the Standard Penetration Test, and provides similar data. One unusual characteristic of the BEEP test in contrast to the Standard Penetration Test is that the higher the consistency of the soil, the less time is generally required to perform the test. This is in opposition to the time requirement relationship of the Standard Penetration Test, in which the harder or denser the soil, the longer the duration of the field test.

Shallow Footing Inspection: The BEEP has also been utilized in evaluating existing footing excavations to verify that the design bearing pressure is available to support the foundation on the underlying soils. In this regard, the BEEP was used to replace the auger

and dynamic penetrometer method of footing inspection, which is widely used within the Southeast. The auger and penetrometer or "A&P" method, consists of hand augering to predetermined depths, stopping the hand augering process, and utilizing a drop hammer to advance a cone penetrometer attached to a steel rod. The "A&P" was originally designed and calibrated to correlate with Standard Penetration Test (SPT) data.[2] Correlation graphs are used to correlate A&P data to SPT data. Allowable bearing pressure correlations with Standard Penetration Test data have been developed by a number of investigators.

In utilization of the Bore Hole Electronic Earth Penetrometer for footing inspections, a hand auger was utilized as in the "A&P" test procedure. In the BEEP test, however, a hole could be augered without interruption. The BEEP was usually lowered to the bottom of the hole, and tests were performed at intervals as the BEEP was raised. This resulted in a more efficient and less time-consuming field inspection for footing bearing capacity determinations. Additionally, the BEEP test procedure was considerably less physically demanding than the "A&P" test.

Soil Parameters Obtained

The one soil parameter obtained from testing with the Borehole Electronic Earth Penetrometer is the horizontal static penetration resistance of the soil. The BEEP method of obtaining static penetration resistance is characterized by a constant rate of penetration which has been built in the most recent BEEP model and the surcharge effect surrounding the cone penetrometer at the time of test to prevent or limit plastic deformation from occurring in the soil near the cone. At this time, the static penetration resistance is the sole soil property obtained. With a modification of the equipment, however, lateral field California Bearing Ratio (CBR) tests could be performed. Since lateral CBR testing has not yet been accomplished, this study will not further evaluate the BEEP system for providing CBR test data.

The static penetration resistance obtained by the BEEP is recorded in the form of the difference in pressure or force required for a measured penetration distance subtracted from the pressure or force required to "seat" the penetrometer at a lesser distance of penetration. This penetration resistance has been correlated with both the Standard Penetration Test (SPT), and the portable dynamic penetration test ("A&P" test). The discussion of the correlation testing which has been performed will be presented in another section of this study. Penetration resistance data obtained from the BEEP are used to estimate strength and deformation characteristics of soils. Even though the data obtained do not relate directly and accurately to shear strength, deformation parameters, or any known single soil engineering property, ranges of strength and deformation characteristics of soils are determinable with this test method. The BEEP test appears comparable in accuracy and utilization to the Standard Penetration Test.

Methodology and Testing

The horizontal static penetrometer assembly is cylindrical in shape being approximately 7.5 centimeters in diameter and 45 centimeters in length as shown in figure 47. The assembly consists of a centrally located, horizontally oriented cone fixed to a rod and piston. The piston is fitted in a cylinder within the penetrometer housing and is equipped with a double action hydraulic system. A recess is provided in the housing so that the penetrometer cone will not protrude from the housing when in the retracted position. The assembly also contains two stabilizers that translate outward and are hydraulically operated separately from the cone assembly. The stabilizers are used to seat the penetrometer housing against the bore hole prior to initiation of penetration testing, resulting in application of a surcharge pressure by the cylindrical housing in the vicinity of the cone. The horizontally oriented cone is 3 centimeters in diameter and has an angle of ninety degrees. Seating and final cone displacement are automatically indicated by a simple electronic system within the housing. Indications of seating and final cone displacement are made at the console by both a visual

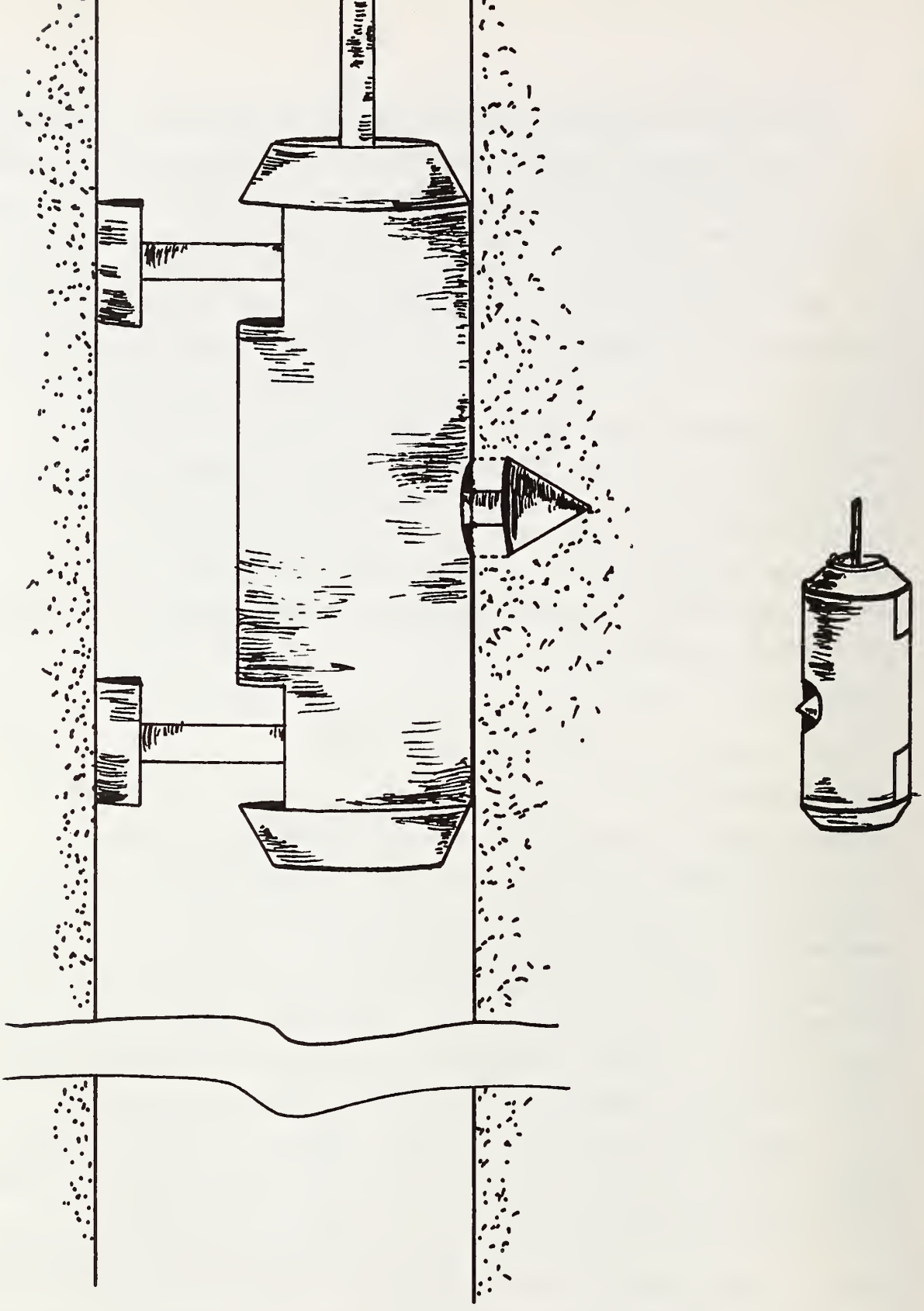

Figure 47 - BEEP Apparatus
2nd Generation

indicator light and, at the same time, an audible noise ("beep"). The hydraulic pressure supply used in controlling the cone and stabilizers is provided by an accumulator. The accumulator is recharged by means of a hand operated hydraulic pump system.

After the bore hole has been advanced for its entire depth, the horizontal static penetrometer assembly is inserted in the hole to the desired test elevation. The two stabilizers are then expanded laterally to seat the device against the side of the hole at a predetermined surcharge pressure. This surcharge pressure is maintained for a period of one minute. The cone penetrometer is than forced horizontally into the soil at a uniform rate of approximately .02 centimeters per second. The penetration resistance measured is defined as a difference in pressure on the penetrometer cone at penetrations of 1.6 centimeters (seating penetration) and 2.9 centimeters (final penetration). Elimination of the seating penetration resistances reduces the effect of soil disturbance near the hole periphery. An indicator light goes on and a buzzer "beeps" at both seating and final penetration, the cone is retracted into its housing, the stablilzers are retracted, and the housing may be raised or lowered to the next desired test elevation. After testing is completed the pressure differentials are computed from the test readings, and Standard Penetration Test (SPT) equivalents can then be determined with a simple single curve nomograph (Figure 48.)

An outline test procedure is shown in the following operational sequence:

1. Lower the BEEP housing to the desired testing depth.
2. Expand the stabilizers to engage the wall of the hole.
3. Maintain a predetermined surcharge pressure on the stabilizers for one minute.
4. Actuate the cone penetrometer.
5. Record the initial hydraulic pressure reading at the sound of first "BEEP", and record the hydraulic pressure reading at the sound of the second "BEEP".

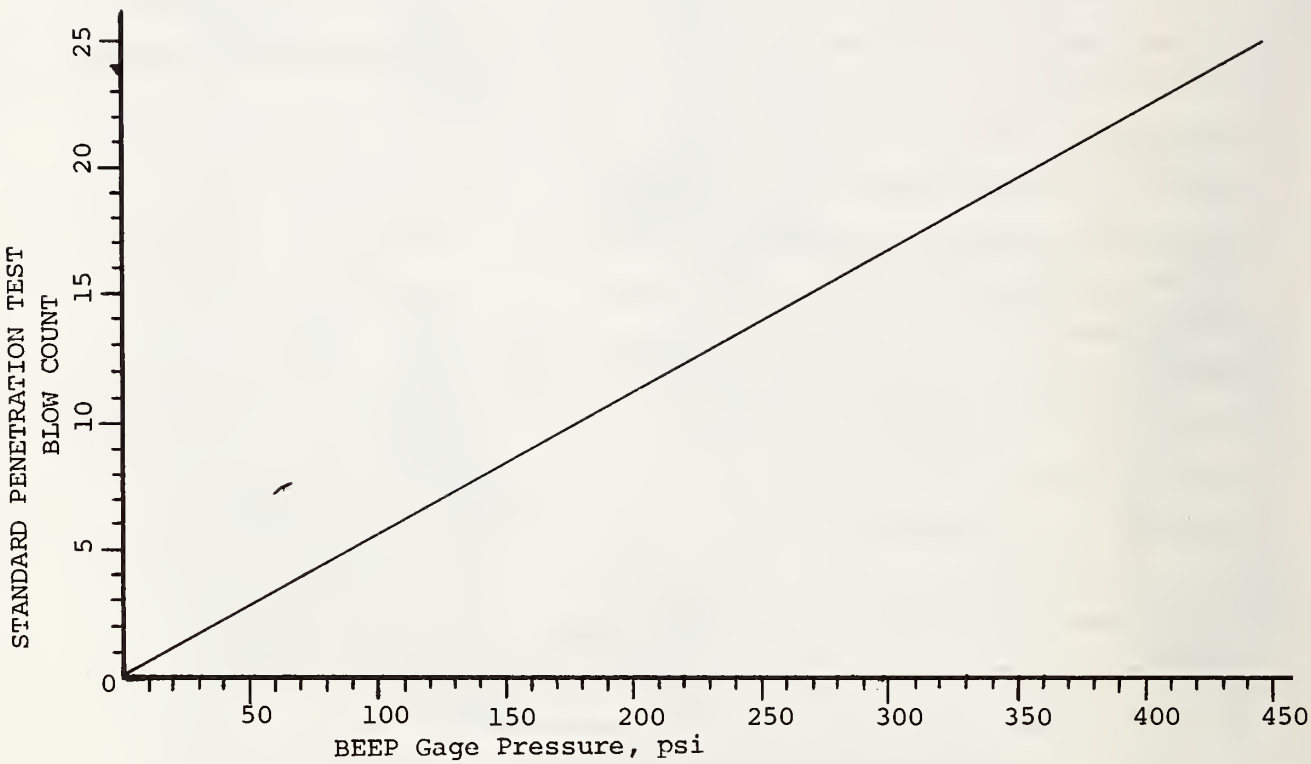

Figure 48 - BEEP Nomograph - 2nd Generation Unit

6. Compute the pressure differentialand refer to a simple single curve nomograph to determine the Standard Penetration Test (SPT) equivalent.
7. Retract the cone penetrometer and the stabilizers, raise (or lower) the unit to the next testing level, and repeat the above sequence.

The test parameters have been successfully obtained in a wide variety of soil types and conditions, primarily in the States of Georgia and South Carolina. Soil types and conditions which have been successfully tested with the BEEP method include: well-compacted micaceous sandy silt fills; soft uncompacted micaceous silt fills; soft alluvial silty clays; lake deposited sands and silts; stiff residual sandy micaceous silts; stiff residual clayey silts; and coastal plain clays and sands of varying consistencies.

In the initial correlation testing, six test sites in the State of Georgia having a relatively wide variation in soil types were utilized to obtain field correlation with dynamic penetration tests.[3] Two of these sites consisted of well-compacted micaceous sandy silt fill materials. At two other sites, residual soils consisting of stiff sandy micaceous silts and stiff clayey silts were tested. The soils tested at the remaining two sites included lake deposited sands and silts, soft alluvial silty clays, and soft uncompacted micaceous silt fills. Subsequent use of the BEEP to provide parameters for design usage expanded the number of test locations and types of soils evaluated.

Accuracy

The accuracy of BEEP static penetration data can be evaluated by comparing BEEP test data with that of the Standard Penetration Test and also with the dynamic cone penetration resistance data. The statistical accuracy of BEEP data correlated with both the Standard Penetration Test and the dynamic penetration test was very good.

The results of 190 BEEP penetration tests yielded a standard error of approximately 1 blow per foot (0.96 blows per foot) for soils with dynamic cone resistances of 1 to 16 blows per foot. Please refer to Figure 49. The standard error for soils with consistencies from 16 to 22

blows per foot was higher than 1, but less than 15% of the blow count. The accuracy of correlation can be expected to be reduced in higher consistency soils above 22 blows per foot, but in this range quantitatively exact SPT equivalents are less meaningful than in moderate to low consistency soil levels. In other words, whether a soil is 45 blows per foot or 53 blows per foot is unimportant, but the difference between a 2 blow per foot soil and a 4 blow per foot soil can be critical.

In very soft or very loose soils, the stabilizer pressure can actually cause a bearing capacity type failure of the soil, and can therefore prevent completion of meaningful BEEP tests. On the other hand, the occurrence of such a bearing capacity failure by the stabilizers is readily discernable by an experienced BEEP operator. To date, such a stabilizer-caused bearing failure has uniformly meant that the soil was less than one blow per foot in consistency. Although the BEEP test itself was not performed in such cases, failure of the stabilizers has therefore proven to be an indirect indication of exceptionally weak and highly deformable materials.

A second limitation is in the ability of the commercial model BEEP to quantify penetration resistance in very hard and very dense soils with penetration resistances above 60 blows per foot. The BEEP, as it is presently designed can determine that the soil tested has a consistency in excess of 60 blows per foot, but cannot ascertain the magnitude in excess of 60 blows. An ability to continuously monitor penetration translation (such as by strain gage) rather than discrete indicators at seating and final penetration distances could enable a BEEP to estimate the consistency of very hard and very dense soils. At this time, the manufacturer does not believe this improvement is warranted.

Limitations Imposed by Site Conditions

Cave-in of the bore hole in certain soil types: As is true with most in-situ soil testing instruments utilized in bore holes, the bore hole must remain open during performance of the test. Cave-ins and sloughing of the soil (squeezing-in) will occur in certain soils and

therefore can hinder or prevent testing. Drilling mud has successfully been used to prevent cave-ins and squeezing-in of weak and compressible soils which are sensitive to this problem.

Bearing capacity failure of very soft and very loose soils: As previously mentioned, exceptionally soft or loose soils may undergo a bearing capacity type failure within the vicinity of the stabilizers caused by stabilizer pressure. This limitation, although it will prevent horizontal static penetration tests from being performed, does alert the operator that the soil being tested is exceptionally weak. That fact alone may be sufficient for engineering judgements such as the necessity to obtain undisturbed samples for subsequent laboratory evaluation testing.

Ground Water: The BEEP is designed for use under water, however, ground water in certain soils can cause cave-in or may change the engineering characteristics of the soil in the vicinity of the hole periphery. Should this occur, it is possible that BEEP tests could result in data which are excessively low in comparison with the undisturbed state of the soil. However, if the soil is tested immediately after a hole has been advanced, reduction in strength of the soil tested should generally not be a problem. Furthermore, use of drilling mud can prevent this occurrence. The BEEP has been used on numerous occasions below ground water. If the hole stays open, no problems are likely to occur.

Soil Properties and Empirical Relationships

Like the Standard Penetration Test and dynamic penetration tests, the BEEP can provide data to evaluate soil properties and to be used in empirical design relationships. Basically, all soil property evaluation information and empirical relationships provided by the Standard Penetration Test and other dynamic penetration tests can potentially be provided by the BEEP. Soil properties used in design which can be derived from data provided by the BEEP are: soil consistency in blows per foot equivalents; ranges of soil strength; and ranges of soil deformation characteristics. Potentially with a modification of the apparatus, the BEEP test concept can be used to provide in-situ California Bearing Ratio (CBR) data.

Empirical design relationships which can be determined from BEEP test data include: correlation with SPT and A&P (and other dynamic or static penetration tests); allowable bearing pressures for shallow footing systems; ranges of relative density in sands; ranges of consistency in cohesive soils; estimation of the angle of internal friction or cohesion in soils; estimation of the in-place shear strength of cohesive soils; estimation of the modulus of compressibility in simple consolidation of sands; estimation of the relative density and cohesion of soils above the ground water table; and estimation of the compressibility and relative density of coarse sands below the water table.

In-House Research and Development

Six test sites located in the State of Georgia, having a relatively wide variation in soil types, were chosen to obtain field correlations with the Standard Penetration Test (SPT) and dynamic penetration test (A&P tests).[3] Two sites consisted of well-compacted, micaceous sandy silt fill material. At two other sites, residual soils consisting of stiff sandy micaceous silts and stiff clayey silts were tested. The soils tested at the remaining two sites included lake deposited sands and silts, soft alluvial silty clays, and soft uncompacted micaceous silt fills.

In this developmental work, three horizontal bore hole penetration tests were performed at the same elevation in each test hole by rotating the penetrometer housing 120 degrees after each test. Sixty-three sets of three tests each and one single test were performed at the sites described, giving a total of 190 horizontal penetration (BEEP) tests. At each corresponding test boring location and depth, a dynamic cone penetration test (A&P test) was also performed.

Briefly, the dynamic cone penetration test consisted of advancing a 3.8 centimeter diameter, 45 degree cone penetrometer a distance of 4.44 centimeters at the bottom of a bore hole by means of repeatedly dropping a 6.79 kilogram steel weight through a height of 51 centimeters.

In addition to the dynamic cone penetration test, Standard Penetration Tests were performed immediately adjacent to 14 of the 64 selected test locations. Approximately one-third of the 190 horizontal penetration tests were performed below the water table.

Comparisons of the horizontal static penetration test data with the vertical dynamic cone penetration resistances are shown in Figure 3. Figure 50 shows the averaged penetration values for each test set for readings of pressure difference as a function of A&P penetration resistance. Figure 51 shows the relationship between the Standard Penetration Test values and the horizontal static penetrometer for 14 average test sets.

The standard error obtained using the average of the three values of final pressure was 0.96 blows for soils with dynamic cone resistances up to 16 blows per foot. A break in this correlation was observed for soils with dynamic resistances above 16 blows. Final pressure readings were obtained only for soils with dynamic penetration resistances less than 22 blows per foot, due to limitations of the hydraulic system at the time this testing was conducted. The apparatus has since been redesigned to increase the capacity to an equivalent of 60 blows per foot.

The standard error between the 14 averaged horizontal cone penetration test sets and the Standard Penetration Test values was 0.91 blows per foot. Included in this correlation were alluvial floodplain soils and compacted micaceous silty sand fills.

The standard error using individual BEEP test data corresponding to single dynamic penetration tests rather than the average of each set of three individual tests was 1.08 blows per foot. Thus averaging the three individual tests performed at each test depth decreased standard error from 1.08 to 0.91, an error reduction of approximately 20%. A further statistical refinement was attempted by selecting and discarding the one test result in each set of three that was furthest from the set average, and recomputing the average of the new set of two tests. This averaging method resulted in increasing the standard error from .91 to 0.96. Thus, the best statistical results were obtained by the averaging of all three tests in each set. This result implies that scatter normally obtained in penetration testing may at least partially be

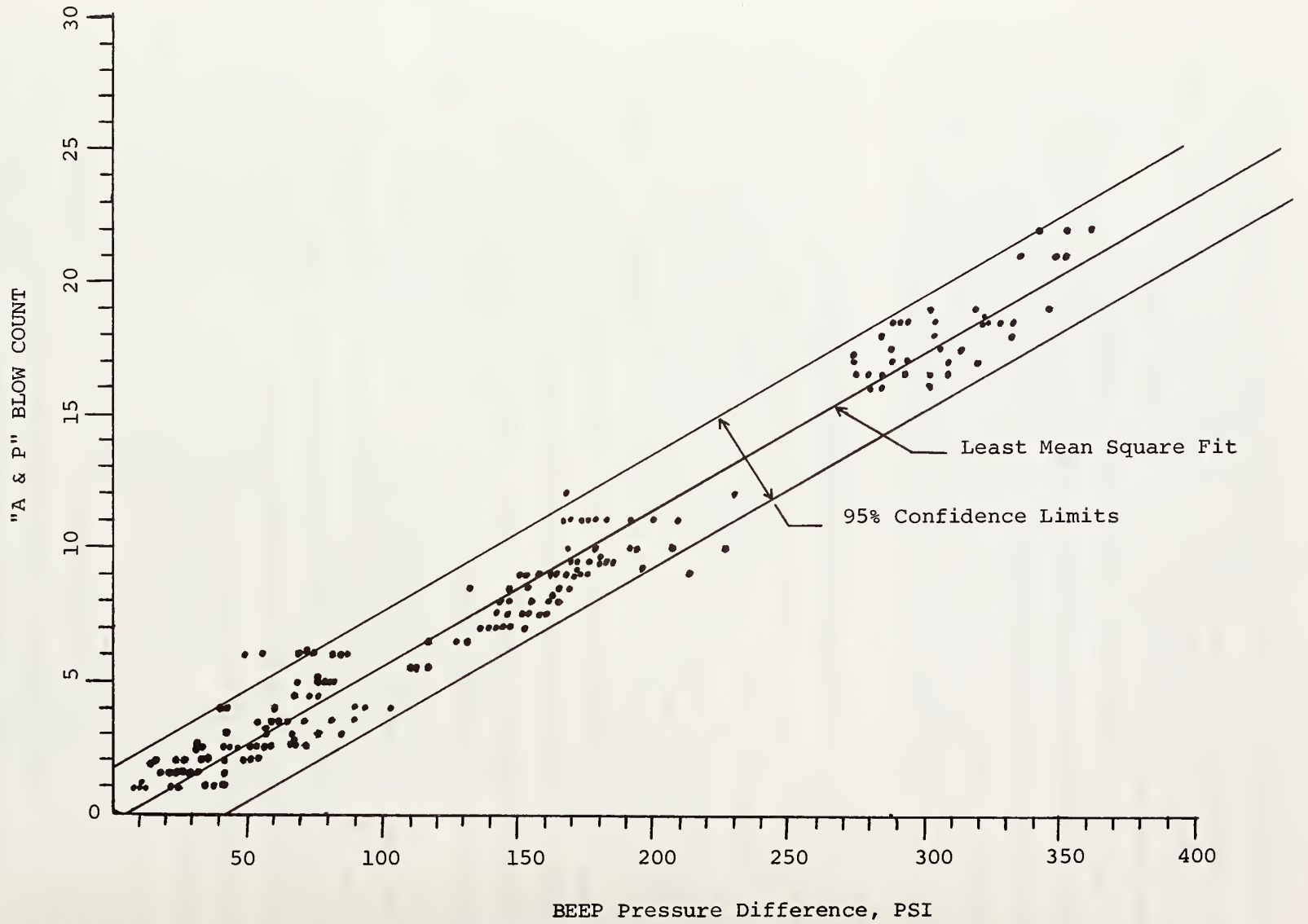

Figure 49 - BEEP Single Test Penetration Resistance vs. Dynamic Penetrometer ("A & P")

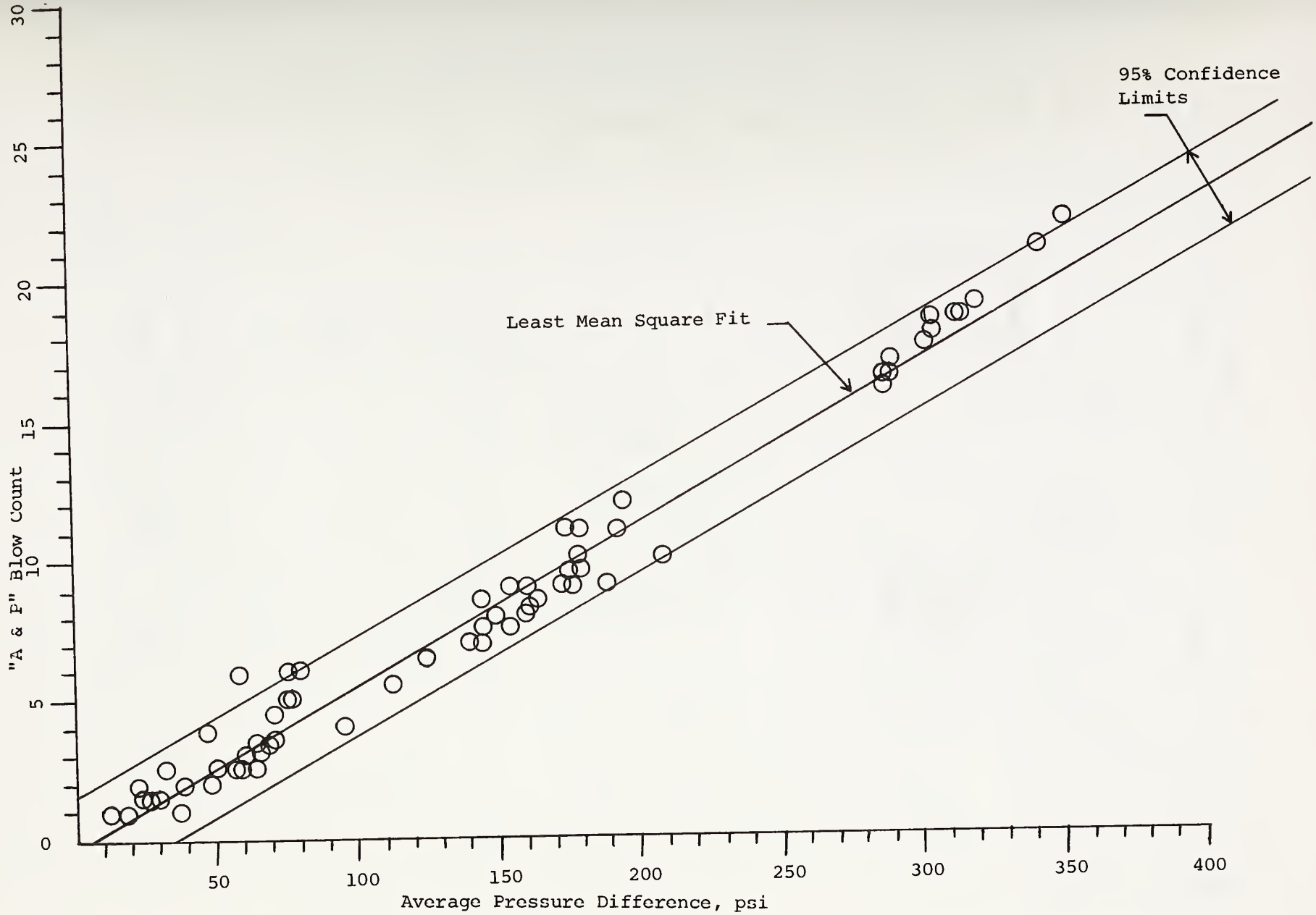

Figure 50 - BEEP Average Penetration Resistance vs. Dynamic Penetrometer ("A & P")

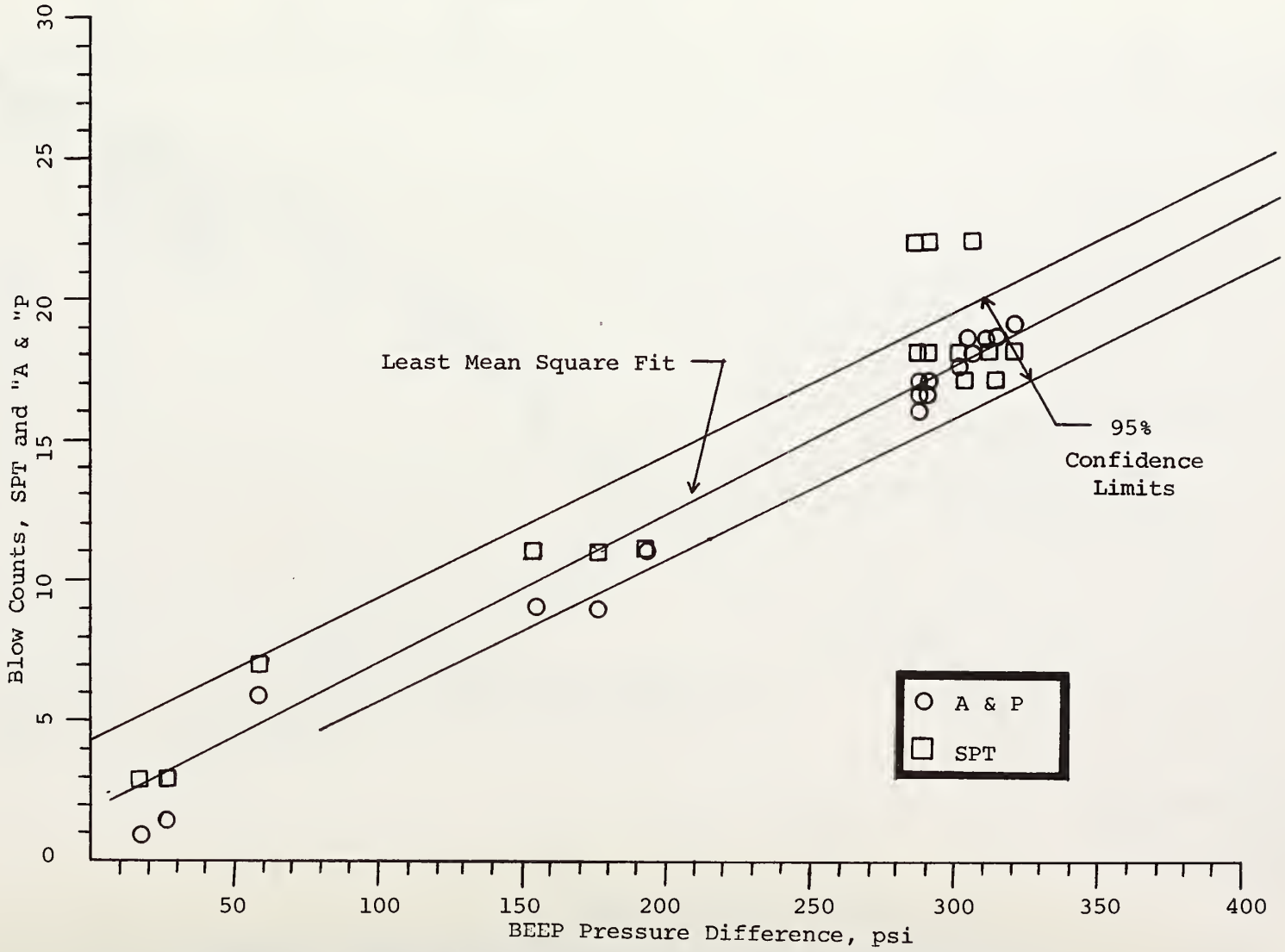

Figure 51 - BEEP data vs. SPT and "A & P" data

attributable to not averaging a sufficient number of tests at the same test location, and therefore not obtaining representative soil resistance values.

In actual consulting usage, the accuracy of a standard error of 1.08 blows per foot was considered adequate. The additional accuracy of performing three tests was not considered justifiable for the additional time and cost. The recommended test method therefore requires only one horizontal test to be performed at each elevation tested.

Significant differences in soil reaction would appear to exist between static and dynamic penetration test methods. Furthermore, certain soils exhibit differences in engineering properties and stress states in the vertical and horizontal directions. As a result, the close correlation obtained between the horizontal static penetration tests and vertical dynamic penetration testing, with a single linear relationship was somewhat surprising.

The high degree of correlation and the single linear relationship appears primarily to be a result of the use of a 52 kN/M^2 surcharge pressure applied to the bore hole wall surrounding the cone. Also, the electrical measurement of the horizontal displacement accurately to 0.01 centimeter reduces operator error in this test and hence also decreases the scatter in test data.

The use of a surcharge pressure reduces the potential of perimeter shear which has been associated with small model footings.[4] The surcharge pressure also exerts stresses in the soil which may tent to override the gravational body forces which are asymmetrical with the horizontal penetrometer. Finally, the surcharge pressure probably reduces the size effect influence of the small cone by causing an increase in effective simulated body forces.[5] It has been found that the scatter of dynamic cone penetration test data was significantly reduced by using effective overburden pressures in analyzing test results. This result indicates an important influence of overburden stress and tends to reinforce the hypothesis that the surcharge pressure applied in the horizontal penetration tests has a significant influence in reducing the scatter of data.

CASE HISTORIES

Case History Number 1: Warehouse Structure Failure Investigation:

A subsurface investigation was performed in August, 1972, of a partially completed warehouse structure within the Piedmont Geological Province in central Georgia.[6]

The structure was of prestressed reinforced concrete design. Exterior column loads were approximately 73,000 pounds; interior column loads were designed to be 126,000 pounds. The concrete slab-on-grade was designed to carry heavy forklift trucks. Floor loads of stacked lumber were estimated to be 750 pounds per square foot spread over an area of 16 feet by 16 feet.

At the time of the investigation, exterior and interior column footings had been constructed within seven column lines. An open trench existed along a column row in a loading dock area. Soils encountered in excavating this trench were very soft and compressible. Because of encountering this very soft soil, a geotechnical engineering consulting firm headquartered in the Atlanta area was requested to conduct an investigation.

The scope of this investigation, as jointly determined by the owner and the consultant, included the following objectives:

(1) To determine the allowable bearing pressures of the soils within stressed zones in both the existing footing areas and proposed areas.

(2) To determine the lateral extent, thickness, and engineering characteristics of the very soft, low density soil zone.

(3) To evaluate the existing foundations from the standpoint of safety and function.

(4) To make recommendations on remedial measures for existing foundations, and design and construction considerations for planned foundations in the future construction area.

The initial phase of the subsurface investigation included the performance of twelve soil test borings utilizing the BEEP to determine static penetration resistance and allowable bearing pressures. Results

of six static penetration tests per boring (borings being 10 feet deep) were presented in interpretive form. Static penetration resistances for design evaluation were determined by selecting the lowest reliable static penetration resistance per boring. These values were correlated with a design correlation curve developed during the initial BEEP research, to correspond to "Standard Penetration Equivalents" and then to allowable bearing pressures. In summary, the process of estimating allowable bearing pressures was:

(a) Auger drilling and obtaining soil cutting samples.

(b) Lowering the BEEP to test the sides of the hole at 2 foot increments of depth.

(c) Obtaining static penetration resistance values (final translation resistance minus seating penetration resistance).

(d) Determining the "design" static penetration resistance as the lowest reliable value per boring.

(e) Correlating static penetration resistance to standard penetration resistance in equivalent "blow counts".

(f) Using allowable bearing pressure correlation curves with the equivalent standard penetration resistances.

Excessively soft soils were found in two borings. Based on this information, supplemental soil test borings using conventional standard penetration testing were performed. The BEEP testing had been accomplished in a period of 2 days. The static penetration data from the BEEP testing had isolated a very soft zone of highly micaceous and compressible silt within portions of the site. The Standard Penetration Tests were utilized in lieu of additional BEEP tests for several reasons: depths were required which exceed the 22 foot depth limitation of the particular BEEP apparatus used in that study; and undisturbed thin wall tube samples were required for laboratory consolidation and triaxial shear tests as a check on the BEEP data, since this was the first consulting project in which the BEEP was used.

Standard Penetration Tests verified the accuracy of BEEP tests. Of particular interest were the soft and very soft zone soils encountered in

three static penetration borings. Laboratory consolidation tests and triaxial shear tests were performed on undistrubed samples. Results were used in analytical calculations of bearing capacity and estimated settlement. Remedial measures utilizing deep foundations in lieu of certain shallow footings were developed. A description of the subsurface conditions is presented below.

The site of the lumber warehousing facility is located east of Montreal Circle and south of the Seaboard Coastline Railroad main track in DeKalb County, Georgia. At the time of the investigation, wall and interior column footings had been constructed within the main warehouse facility extending from column line 8 to column lines 14. Erection of columns and prestressed concrete wall and roof sections had not commenced at the start of the field investigation. An open trench existed in the loading dock area. Soils encountered in excavating this trench were very soft and wet. Attempts had previously been made to dry out this soil by lowering the water table through a "french drain" type of dewatering system.

All borings encountered residual soils. "Residual" denotes soils which have been formed by the in-place weathering of crystalline rock. The residuum at this site varied considerably in appearance and character, but was generally a firm to very stiff fine sandy micaceous silt with a zone of very soft to firm highly micaceous soil in the central and eastern portions of the site. This soft to firm zone varied in thickness from a maximum of approximately 17 feet in the northeastern corner of the site, tapering to approximately 4 feet in the northwestern corner of the site. In the southern direction, the soft to firm highly micaceous silt zone averaged approximately 7 to 8 feet in thickness, but was sandwiched between overlying and underlying stiff residual soils.

In the one deep boring (b-2), the soil consistency gradually increased with depth. Very hard soils and partially weathered rock were found at a depth of approximately 65 feet.

In summary, the BEEP data proved to be an accurate and rapid method of locating the soft to very soft, highly micaceous zone. Supplemental

standard penetration testing further verified the accuracy of the low static penetration resistance data. This information was necessary to determine approximate lateral extent and thickness of the unsuitable zone. Results of BEEP tests and Standard Penetration Test data were then used to select locations in which to obtain undisturbed samples for laboratory testing and analytical evaluations. These data and analyses resulted in quantitative remedial recommendations.

Case History Number 2: Four Story Office Building Foundation Investigation

A subsurface investigation and engineering evaluation was performed in November, 1971, within the site of a planned medium rise office structure. The planned building was four stories in height and 120 feet by 240 feet in plan dimensions. Maximum column loads were anticipated to be 600,000 pounds. The structure was of precast concrete exterior and reinforced cast-in-place concrete frame construction.(7)

Six soil test borings utilizing the BEEP apparatus to provide static penetration resistance were performed. Borings were advanced to refusal in five of the six locations. Refusal to the mechanical drilling process was encountered at depths varying from 7 feet to 43 feet. One soil test boring was terminated at a depth of 50 feet. In addition, three auger borings were advanced, two to auger refusal, and one to a depth of 50 feet.

Raw data from the BEEP tests are presented in Table 12. These data were correlated to standard penetration equivalent values with the correlation curve developed in the initial BEEP research. Results of SPT equivalents, along with descriptions of soil strata, undisturbed sample locations, and ground water encountered, were plotted on boring logs.

The general subsurface profile included still to very hard sandy micaceous silts, with generally increasing consistencies with depth. An exception to this pattern was found in one boring area, (B-9) where a lower consistency silt was encountered from 12 to 22 feet below the existing ground surface. Pinnacles of seamy rock and weathered rock

TABLE 12

BEEP Penetration Resistances

Boring Number	Depth, feet	BEEP DATA: Seating Pressure, on	Seating Pressure, off	Final Pressure	Difference	Blow Count Equivalent
B-1	5	200	250	390	190	11
	10	500	---	---		20+
	15	330	500	---		20+
	20	320	500	---		20+
	25	360	500	---		20+
	30	410	500	---		20+
	35	500	---	---		20+
	40	500	---	---		20+
B-3	2.5	295	440	---		20+
	5	500	---	---		20+
	10	500	---	---		20+
B-4	5	375	400	---		20+
	10	340	430	---		20+
B-6	5	370	500	---		20+
B-9	5	500	---	---		20+
	10	290	370	---		
	15	210	280	350	140	8
	20	220	270	350	130	8
	25	460	---	---		
	30	295	380	490	195	11
	35	295	400	---		20+
	40	250	410	---		20+

Note: Pressure numbers are in gauge psi. To convert to kN/m^2, multiply by 6.89.

existed within the stiff to very hard soil matrix. Ground water was found only in the deeper borings, at a depth of 37 to 38 feet from the existing ground surface.

Six different foundation types were evaluated as a result of the auger borings and BEEP static penetration data. These included:

(1) Shallow spread footings bearing on residual soils.

(2) Shallow spread footings bearing on rock.

(3) Drilled piers bearing on residual soils.

(4) Drilled piers bearing on rock.

(5) Flexible mat.

(6) Rigid mat.

Undisturbed samples were obtained, and consolidation tests were performed in select zones.

Settlement calculations indicated that differential settlements of approximately one and one half inches would be anticipated if shallow foundations bearing on residual soils were selected as the foundation system. The maximum allowable differential settlement for this structure was 0.5 inches; therefore this alternative was considered unsafe and was rejected. A supplemental investigation which included three soil test borings utilizing Standard Penetration Tests and rock coring, was performed to evaluate the economics of shifting the building location 50 feet.

Drilled and belled piers bearing on residual soils were also determined to be unsafe. Both of the mat alternatives were determined to be uneconomical. The final recommendation was to employ a combination of shallow footings bearing on rock and weathered rock, and drilled piers bearing on relatively high consistency soils existing at depths of 30 feet or greater. An end bearing pressure of 10,000 pounds per square foot was used in the drilled pier design, with 15,000 pounds per square foot recommended for weathered rock footing bearing pressures.

The foundations were installed under geotechnical engineering observation, and the building has performed well.

In summary, the BEEP data, being relatively rapid and inexpensive to obtain, allowed a reasonable coverage within a highly erratic subsurface situation. A lower consistency "firm" zone of sandy micaceous silt was identified by use of the BEEP data. An undisturbed sample of this soil was obtained and utilized for subsequent consolidation testing and settlement calculations.

Case History Number 3: Apartment Complex, Richland County, South Carolina:

A subsurface investigation and engineering evaluation was performed for a planned apartment complex to be located in a five acre site within alluvial and coastal plain soils of Richland County, South Carolina.(8)

The structures were planned to be two stories high, with no basements. Load bearing walls were anticipated for the foundations. Estimated wall loads were on the order of 2000 pounds per linear foot.

Twelve soil test borings were advanced, ranging in depth from a minimum of 12 feet to a maximum of 20 feet. Borings less than 15 feet deep terminated in material causing refusal to be mechanical drilling equipment. Table 13 shows results of static penetration testing, utilizing the BEEP method. Boring logs were developed to show subsurface conditions encountered. Figure 52 is a boring location plan.

The study concluded that subsurface conditions within approximately 40% of the site were not suitable for the proposed construction without taking positive remedial measures. Soft upper soils, high ground water and future flooding hazards necessitated remedial measures. The unsuitable area consisted of a relatively level floodplain which had been formed by a creek running near the eastern border of the site. Two general methods of stabilizing the low consistency soils were developed. The first was undercutting and removal of unsuitable soils, replacing them with suitable, properly compacted fill materials. The alternative solution was to surcharge the low consistency area, and remove a portion of the surcharge material after preconsolidating the underlying compressible soils.

TABLE 13

BEEP Penetration Resistances

Boring Number	Depth, feet	Seating Pressure, on	Seating Pressure, off	Final Pressure	Difference	Blow Count Equivalent
B-1	5	110	115	220	110	6
	10	150	240	600+	---	18
B-2	5	140	180	280	140	8
	10	170	300	600+	---	21
	15	450	540	600+	---	33
B-3	5	190	300	490	300	17
	10	195	280	600+		22
	15	265	380	600+		28
	20	360	600	---		27
B-4	5	200	330	395	175	10
	10	395	580	---		30
	15	270	390	---		22
	20	160	190	450	290	17
B-5	5	160	175	280	120	7
	10	225	345	600+		17
	15	11 ft cave-in				
B-6	5	145	170	400	255	15
	10	315	580	---		25
	refusal 12 feet					
B-7	5	180	195	485	305	18
	10	180	220	330	150	9
	15	100	100	110	10	2
B-8	5	400	600+	---		
	10	600	---	---		
B-9	5	95	100	140	45	3
B-10	5	110	145	260	150	9
	7	600+				
B-11	5	115	130	140	25	2
	10	135	230	600+		18
B-12	5	185	230	360	175	11
	9	340	595	---		26

Note: Pressure numbers are in gauge psi. To convert to kN/m^2, multiply by 6.89.

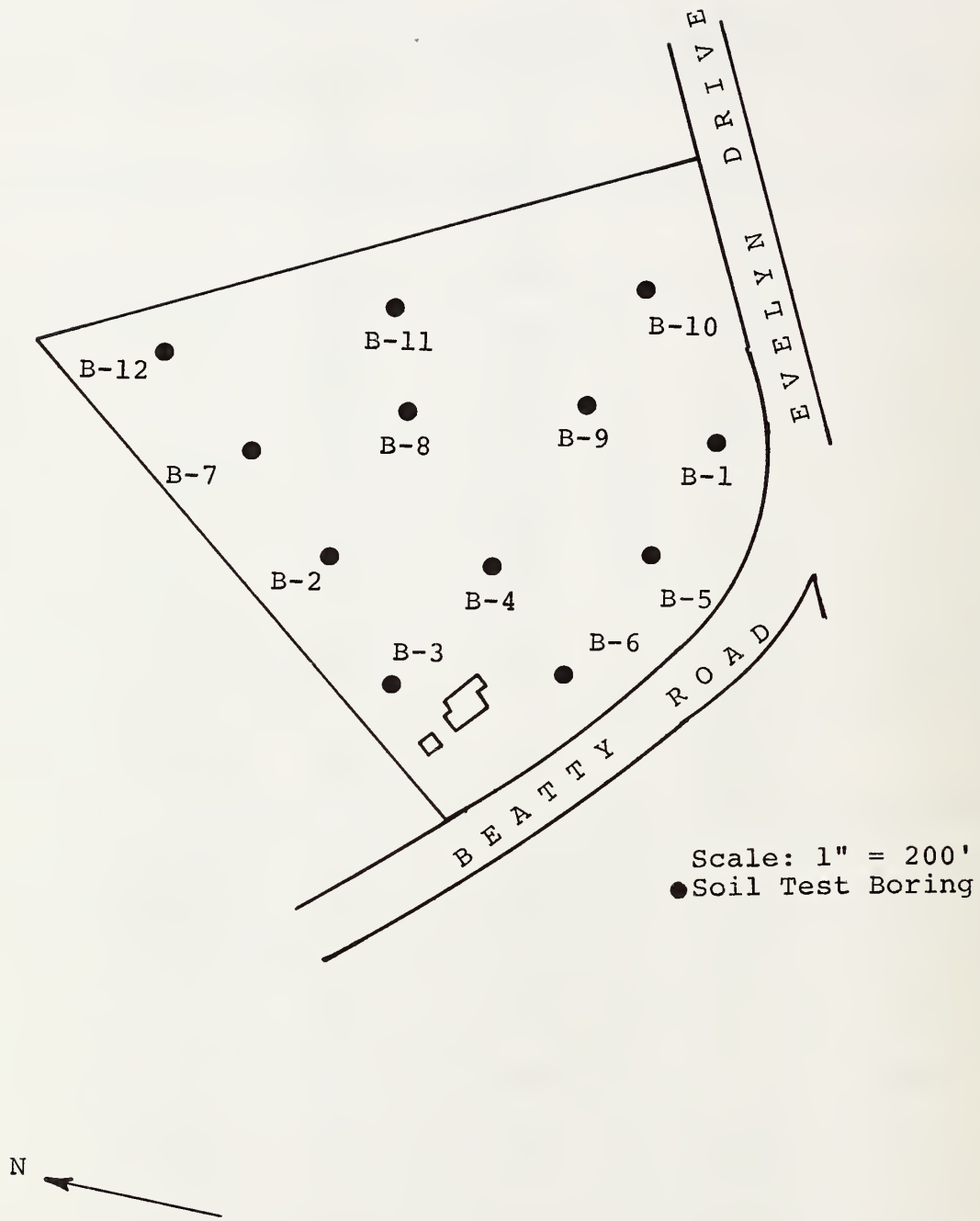

Figure 52 - Boring Plan - Apartment Complex, Richland County South Carolina.

As a result of the data obtained and the recommendations developed, the site was not purchased by the client for whom the investigation was performed, nor was it developed.

This study illustrates the use of BEEP data in poor soil, high ground water, low consistency areas where access to normal drilling equipment may be impossible.

Case History Number 4: Education Facility Subsurface Investigation

A subsurface investigation and engineering evaluation was performed in January 1973, on a site within the coastal plain geologic province of Georgia. A two story classroom building structure was planned to be supported by shallow interior footings and continuous wall footings. An adjacent lobby and lecture hall was also planned. Maximum interior column loads in the two story portion of the complex were 140,000 pounds, with interior columns of 65,000 pounds. Footings in the one story area varied from 30,000 pounds to 50,000 pounds, with maximum wall footing loads of 1800 pounds per linear foot.[9]

Below a veneer of vegetation and topsoil averaging approximately 6 inches thick, the borings generally encountered sandy clay and clayey sand soils with a wide variation in consistencies and subsurface stratigraphy. Within the proposed two story classroom building area, sandy clays having only a firm consistency were found in several boring areas. A very soft sandy clay zone was found in both borings B-1 and B-2 within the proposed lecture hall structure. This very soft zone was approximately 5 feet thick and ranged in elevation from a maximum of approximately 288 in the area of boring B-2 to a minimum elevation of approximately 278 in the area of boring B-1.

Thirteen soil test borings and one auger boring were performed at the site at the approximate locations shown on the included boring plan, Figure 53.

Two undisturbed samples were obtained for possible use in supplemental laboratory testing and analysis. One sample was obtained in a representative firm clay zone within the two story building area. A second sample was obtained within the soft clay zone in the lecture hall area; however this sample was in such a soft condition that it was not suitable for subsequent laboratory testing.

Soil sampling and penetration testing were performed in accordance with ASTM specification D4156-65, supplemented by horizontal static penetration testing, and by ASTM specification D1557. Static penetration data are presented in Table 14.. All soil test boring data, including soil descriptions, were developed and presented on boring logs. From an analysis of the static penetration resistance, soil types, and ground water conditions, the study concluded that the soils within the two story portion of the proposed complex, although not of high consistency, would be suitable to support shallow foundations designed for a maximum bearing pressure of 2000 pounds per square foot. Special foundation measures were recommended within the lecture hall area because of a soft to very soft clay zone existing in the northern portion of the proposed lecture hall. The study outlined four alternative solutions:

(1) Relocate the buildings approximately 50 feet to the south
(2) Relocate the buildings approximately 25 feet to the south and undercut the soft soil zone, replacing it with compacted structural fill.
(3) Do not change the building locations, but undercut the soft soil zone and replace it with compacted structural fill. This alternative would require positive dewatering methods.
(4) Support all column loads for the lecture hall structure on piles, and the floor slab on a structural slab.

The second alternative, to relocate the building approximately 25 feet, and undercut and replace the unsuitable soils, was selected by the

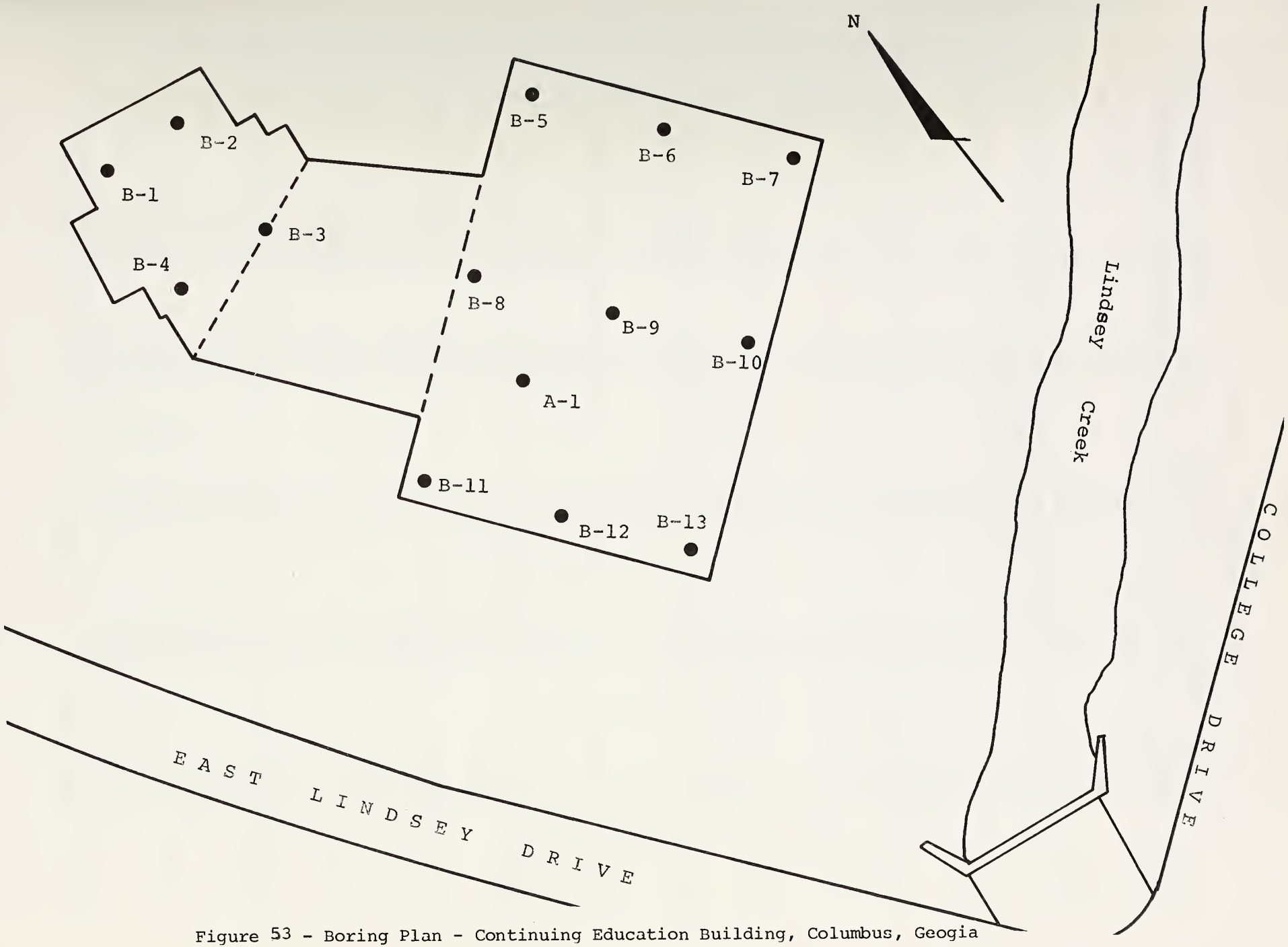

Figure 53 - Boring Plan - Continuing Education Building, Columbus, Geogia

TABLE 14

BEEP Penetration Resistances

Boring Number	Depth, feet	Seating Pressure, on	Seating Pressure, off	Final Pressure	Difference	Blow Count Equivalent
B-1	5	180	250	390	210	12
	10	100	100	100	0	0
	15	100	100	180	80	5
B-2	5	Dynamic penetration resistance,blows/foot				1
	10	Dynamic penetration resistance,blows/foot				14
	15	Dynamic penetration resistance,blows/foot				25
	20	Dynamic penetration resistance,blows/foot				42
B-3	5	250	330	600	350	20
	10	180	240	540	360	20
	15	130	165	250	120	7
	20	330	380	600	270	15
B-4	5	250	285	350	100	6
	10	200	250	385	385	22
B-5	5	205	250	310	105	6
	10	310	370	600	290	17
	15	130	180	330	200	12
	20	170	230	545	375	21
B-6	5	260	---	490	230	13
	10	365	600+	---	---	26
	15	180	250	370	190	11
	20	230	280	420	190	11
B-7	5	225	255	330	105	6
	10	240	310	525	285	16
	15	510	600+	---	---	36
	20	100	110	210	110	6
B-8	5	340	465	600	---	24
	10	90	115	270	180	10
	15	300	350	490	190	11
	20	100	150	305	205	12
B-9	5	200	265	335	115	7
	10	100	100	275	175	10
	15	225	270	345	120	7
	20	265	390	600+	---	21
B-10	5	200	240	310	110	6
	10	140	180	540	400	23
	15	245	300	420	175	10
	20	160	230	500	340	19
B-11	5	210	330	600+	390+	21
	10	170	210	300	130	7
	15	140	185	250	110	6
	20	150	230	350	200	11
B-12	5	185	250	500	315	18
	10	185	220	600+	415+	22
	15	120	125	290	170	10
	19	230	300	500	270	15
B-13	5	350	450	600+	250+	23
	10	370	600	---	---	24
	15	175	230	340	165	9
	20	110	140	320	210	12

Note: Pressure numbers are in gauge psi. To convert to kN/m^2, multiply by 6.89

client. The structures were constructed and have performed adequately. This project illustrates the ability of BEEP data to assess subsurface conditions which have a wide variation in consistency and subsurface stratigraphy. Soft and very soft zones were quickly and positively identified.

Case History Number 5: Warehouse Facility Subsurface Investigation and Engineering Evaluation.

In February and March, 1973, a subsurface investigation and engineering evaluation was performed on a site to support a proposed tilt-up wall warehouse. Wall loads were on the order of 2000 to 3000 pounds per linear foot, and interior column loads were on the order of 60,000 pounds. The site is within the Central Piedmont Geological Province in Georgia.[10]

Nine soil test borings were performed in accordance with ASTM Specification D1452-65, supplemented by horizontal static penetration testing. Resulting static resistance data are shown on Table 4. Test boring records show specific soil descriptions and correlated resistance information.

The general subsurface profile consisted of veneer of vegetation and topsoil averaging approximately 6 inches thick, underlain by firm to hard sandy micaceous silts and clayey silts with zones of quartz fragments.

The conclusions reached from this study included the evaluation that the virgin residual soils were suitable to provide an allowable bearing pressure of 3000 pounds per square foot for shallow foundations.

This project represents a somewhat typical piedmont geological province site, in which no poor or unsuitable soils were found to exist. However, a somewhat marginal, relatively thin "7 blow per foot" zone did exist, which was acceptable for the recommended bearing pressure. No laboratory testing, other than a physical examination of distubed samples obtained during the soil test boring operation, was performed. Figure 54 and Table 15 provides information regarding the data obtained in this investigation. Additionally, boring logs were prepared for the client. The structure was constructed under the observation of a geotechnical engineer. No unforeseen problems occurred, and the performance of the building has been satisfactory.

TABLE 15

BEEP Penetration Resistances

Boring Number	Depth, feet	Seating Pressure, on	Seating Pressure, off	Final Pressure	Difference	Blow Count Equivalent
B-1A	4	220	330	500	---	15
	8	450	500+	---	---	32
	12	140	200	270	130	7
B-2	8	Dynamic penetration resistance				10
	11	Dynamic penetration resistance				11
B-3	4	305	360	500+	---	22
	8	355	475	500+	---	25
	12	200	235	310	110	7
B-4	3	150	210	340	190	11
	5	295	390	500+	---	21
B-5	4	200	300	500+	---	20
	10	200	275	400	200	11
	15	280	390	500+	---	20
B-6	4	230	335	500+	230	20
	8	220	290	420	200	11
	12	140	210	295	155	8
B-7A	2.5	500+	---	---		30+
	5	500+	---	---		30+
	8	465	500+	---		33
B-8	3.5	195	220	495		17
		150	170	500+		20
B-9	4	275	395	500+	---	20
	8	245	320	420	175	10
	12	175	240	350	175	10

Note: Pressure numbers are in gauge psi. To convert to kN/m^2, multiply by 6.89.

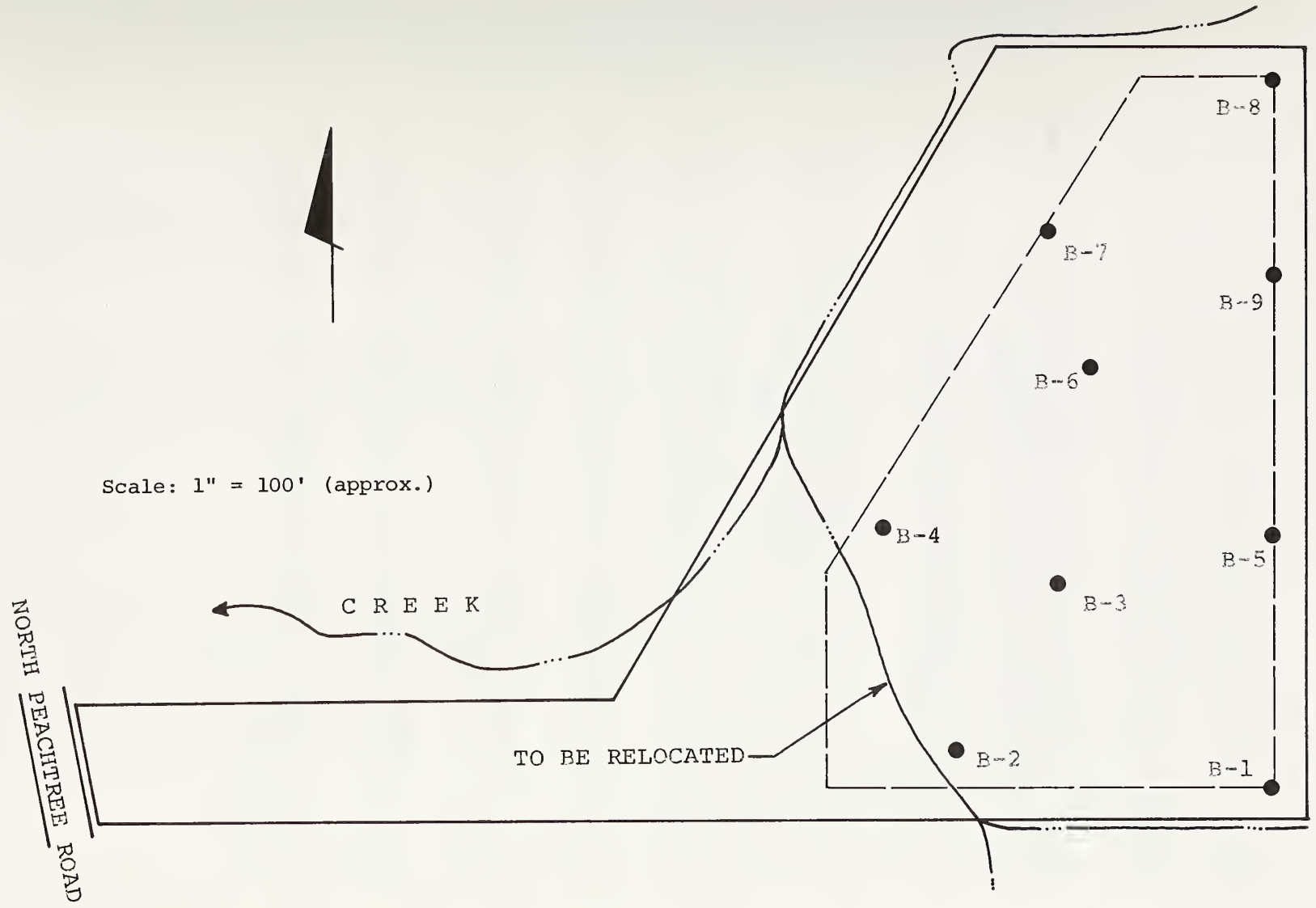

Figure 54 - Boring Plan - Shaheen Warehouse, DeKalb County, Georgia

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

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Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

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This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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